San Fernando, California, Earthquake of February 9, 1971

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Scientific Coordinator

In Three Volumes

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Volume I

EFFECTS ON BUILDING STRUCTURES

Part A. Introduction and Buildings

Part B. Buildings *continued;* Soils and Foundations

Volume II

UTILITIES, TRANSPORTATION, AND SOCIOLOGICAL ASPECTS

Volume III

GEOLOGICAL AND GEOPHYSICAL STUDIES

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VOLUME I

Effects on Building Structures

Part A

INTRODUCTION

BUILDINGS

Foreword

The San Fernando earthquake of February 9, 1971, although moderate in energy release and in amount of surface rupture, led to postearthquake studies that provide much new data and information concerning the effects of an earthquake on multitype building structures—high-rise buildings, low-rise commercial and industrial buildings, dwellings, schools, hospitals, and detention facilities; on the operations and services of public utilities and transportation industries; on human reactions and response to an earthquake emergency in a metropolitan area; on engineering problems related to soils and foundations; and on man's knowledge about and adjustments to the seismic, geologic, and geodetic features of the physical environment. The earthquake occurred in an area where in recent years instrumentation had been installed to make detailed studies of the regional seismicity and the response characteristics of multitype structures to ground motions caused by strong nearby earthquakes. The substantial increase in number of strong-motion seismographs throughout the Los Angeles metropolitan area, and the valuable recordings obtained during this earthquake, will provide the basis for future design of earthquake-resistant structures and timely enactment of improved safety codes.

The number of experienced engineers and scientists in the earthquake area, who were immediately available to conduct studies and investigations in their specialized fields, was fortuitous. Many had gained personal experience from studies of the 1964 Prince William Sound, Alaska, earthquake; the 1966 Borrego Mountain, California, earthquake; and the 1967 Santa Rosa, California, earthquake. For the first time, social scientists were immediately available to observe first-hand reactions and to study postearthquake adjustments. Also fortuitous was the existing active contract agreement between the National Oceanic and Atmospheric Administration (NOAA) and the Earthquake Engineering Research Institute (EERI), which made mobilization of the postearthquake investigations possible within a time span of several hours to a few days. The response of EERI and the Structural Engineers Association of California (SEAOC) has made it possible, in these volumes, to document the engineering and scientific aspects of an earthquake, both in quality and quantity, in a manner never before possible.

Karl V. Steinbrugge, Professor of Structural Design at the University of California, Berkeley, and Head of the Earthquake Department of Insurance Services Office, San Francisco, served as the liaison representative between EERI and NOAA. C. Martin Duke, President of EERI, is Professor of Engineering at the University of California, Los Angeles. They are gratefully acknowledged for the design and procurement of engineering studies in Volumes I and II.

NOAA'S National Ocean Survey (NOS)—the agency of the Federal Government having responsibility for investigations and reports on seismology at the time of the earthquake—initiated NOAA field investigations and implemented the contract agreement with EERI to bring together and to publish detailed findings of postearthquake field investigations and studies. This phase of postearthquake field activity and compilation of research papers was completed by the NOS Office of Seismology and Geomagnetism–Seismology Division prior to transfer of these NOS units to NOAA's Environmental Research Laboratories. The extensive field investigations by the NOS Office of Geodesy and Photogrammetry also are acknowledged.

Leonard M. Murphy

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Editors' Preface

Each volume of this three-volume set differs from the other volumes in organizational structure. This variation, which is readily noted in the grouping of papers and special introductions to groups of papers, reflects the postearthquake investigation activities and responsibilities of the various subcommittees of the NOAA/EERI Earthquake Investigation Committee.

Organization of Volume I

Volume I describes the effects of the earthquake upon building structures and upon soils and foundations as these relate to buildings. It is bound in two parts: Part A. Introduction and Buildings; and Part B. Buildings (continued) and Soils and Foundations. Pages in Volume I are numbered consecutively through Parts A and B. The greater part of the volume is devoted to reports on specific buildings. These are treated in the section on "Building Reports," where they are grouped broadly by earthquake-resistant and nonearthquake-resistant design and further classified according to type of structure and use category, geographic location, and engineering firms that conducted the investigations. Within "Building Reports" there also are papers that treat specific topics—problems in design or type of construction, analytical procedures used in the investigations, special analyses, summaries, conclusions, and recommendations—and papers that are organizational in scope. The latter introduce groups of papers and outline treatment within respective groups. They begin by indicating the SECTION CONTENTS. The entries under section contents generally are *paper titles, preceded by page numbers* to guide the reader to papers that follow. Sometimes there are *entries without page numbers* that show only organizational relationships among the papers in the group. The overall treatment and grouping of papers in the section on "Building Reports" is shown in the table of contents for Volume I.

Organization of Volumes II and III

Volume II considers utilities, transportation, and sociological aspects. Utilities are divided into energy and communication systems and water and sewerage systems. This arrangement, together with transportation systems and sociological aspects, imparts a four-way subdivision to the volume. Each part has been prepared under the direction of, and is prefaced by the remarks of, a special subcommittee of the NOAA/EERI Earthquake Investigation Committee.

Volume III contains findings of geological and geophysical studies that were conducted after the earthquake. The papers in this volume are grouped in the following categories: classical seismology, surface and subsurface geology, vertical and horizontal geodesy, strong-motion seismology, and geomagnetism.

Disclaimer

The reports contained in these volumes include detailed findings in engineering seismology—particularly, objective evaluations of causes and effects in earthquake damage—and in the seismic and geologic characteristics of the physical environment. Although derived entirely from the analysis of factual data, some of the findings, in a very limited sense, might be interpreted as damaging to individual segments of business and industry or as being subject to other interpretation.

The scientific papers reflect the interpretation and opinions of their authors and do not necessarily represent the viewpoints of the National Oceanic and Atmospheric Administration or the United States Department of Commerce. The United States—while providing for the presentation of these papers in the public interest and for their obvious informational value—assumes no responsibility for any of the views expressed therein. The National Oceanic and Atmos-

pheric Administration's Environmental Research Laboratories, in the interest of fulfilling its statutory functions, offers these volumes as a forum for professional experts in the fields of engineering seismology, geology, and geophysics, and for public officials charged with the administration of emergency preparedness and safety programs.

The National Oceanic and Atmospheric Administration does not approve, recommend, or endorse any proprietary product or proprietary material mentioned in this publication. No reference shall be made to the National Oceanic and Atmospheric Administration, or to this publication furnished by the National Oceanic and Atmospheric Administration, that would indicate or imply—directly or indirectly—that the National Oceanic and Atmospheric Administration approves or disapproves of the use of any proprietary product or proprietary material mentioned herein.

Acknowledgments

Publication of this three-volume set involved the efforts of many individuals. Those who mobilized and conducted the postearthquake field investigations and those who contributed papers are acknowledged throughout the three volumes in special papers that describe those activities. Christopher Rojahn, of NOAA's Seismological Field Survey, San Francisco, Calif.—who served as a NOAA liaison representative with the Earthquake Engineering Research Institute —coordinated the compilation of manuscripts and illustrations and was responsible for the engineering drawings prepared by Criss Bangs, Gary Bankhead, Francesco Cappellari, James Fong, Randy Harris, Diane Herold, Sinan Sabuncuoglu, and Richard Spickard. Among the many persons who assisted in the technical processes of publishing these volumes, the editors express their special appreciation to Lola T. Dees, Lila V. Paavola, John R. Bernick, and Gabriel J. Bren for help in editing the volumes; to Edward W. Koehler, NOAA Publications Officer, and William E. Kusterbeck for printing and production assistance at all stages; and to John L. Cooke of the U.S. Government Printing Office, for designing and guiding the publication through the final stages of production.

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Part A

INTRODUCTION

BUILDINGS

The San Fernando Earthquake

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The San Fernando, California, earthquake occurred at 6:01 a.m. (local time) on February 9, 1971, killed 58 persons, 47 in the collapse of the nonearthquake-resistive Veterans Hospital, and caused over 2,500 hospital-treated injuries in the San Fernando Valley, which had a population of over 1,200,000 at the time of the earthquake. The earthquake's epicenter was in the San Gabriel Mountains, its strong motion lasted about 12 seconds, and its magnitude has been assigned as 6.4 on the Richter scale.

Direct damage to buildings and other structures exceeded half a billion dollars. This amount was divided about equally between public and private property. Most of the severe damage and major losses were along the southern foothills of the San Gabriel Mountains and along a narrow band of surface faulting that runs east-west on the valley floor.

Except for the financial losses, the foregoing statistics are not unduly impressive in terms of a major earthquake disaster. An earthquake of magnitude 6.4 occurs about once every 2 years in California or western Nevada. However, what is impressive, and disturbing, is the type and extent of serious damage to earthquake-resistive buildings, to dams located upstream from densely populated areas, and to public utilities and roadways that are the lifelines of cities. An extrapolation of San Fernando earthquake damage to the level of a maximum credible earthquake in the metropolitan Los Angeles area indicates there would be a major catastrophe, possibly exceeding the Nation's capabilities to make an adequate immediate response to such a disaster.

Of particular concern were the strong motions registered by a strong-motion recording instrument in the heaviest shaken area. These motions were greater than any previously recorded. Although the full interpretation of this instrumental record is controversial, the damage to nearby wood frame dwellings and to hospitals clearly indicates that a new look must be given to building codes.

Clearly it is of vital public concern to learn as much as possible about the scientific and engineering aspects of this disaster. It is important to do this quickly and to apply the lessons learned. The papers in these volumes are a first step in this direction.

Intent of Volumes.—The principal intent of these San Fernando earthquake volumes is to bring together the vast quantity of information obtained during postearthquake field investigations and studies and to provide the best possible engineering and scientific analyses as soon as possible after the event. This short time span is important from a public service standpoint because: (1) the enactment of improved safety standards that require public approval can best be accomplished immediately following such natural disasters, when the public is acutely aware of the problems; and (2) the published results of these studies provide the basis for much needed continuing research work. At this writing, information and findings in these volumes are of particular value to proposed changes in the city of Los Angeles building code, reexamination of the seismic safety of dams in California, action on several bills in the California Legislature, and the planning of several Federal agencies.

The conclusions reached and the data presented in these volumes, however, are by no means all inclusive. Much research must be accomplished to explain some of the observed damage and instrumental data. Indeed, studies by numerous universities under the sponsorship of the National Science Foundation are continuing long-term efforts. We expect that many additional papers, and possibly volumes, about the San Fernando earthquake and its effects are to be published.

The papers in these volumes reflect a multidisciplinary accomplishment in their compilation. The most effective applications of the findings will require continued multidisciplinary efforts. The placing of papers from various disciplines within a single volume provides little assurance that authors or readers will pay attention to papers other than those in their specific specialties. This three-volume set not only contains many papers in earthquake engineering, and representative papers in geology and seismology, but also includes such papers as "Individual and Organizational Dimensions of the San Fernando Earthquake" and "The State Role in Seismic Safety," which consider both human and institutional factors and legislative matters. These reflect significant interplay with other disciplines. Drafts of all papers were made available to members of the advisory groups to the California Legislature's Joint Committee on Seismic Safety (State Senator Alfred E. Alquist, Chairman). A number of authors of papers in these volumes served as advisors and consultants to the Joint Committee on Seismic Safety.

Warning.—The San Fernando earthquake must be taken as a significant and serious warning. At this writing, some of the immediate postearthquake public interest has waned, and the result is that some levels of government show growing lethargy to take strong preventive actions.

A maximum credible earthquake, in either the metropolitan Los Angeles area or metropolitan San Francisco area, might release 1,000 times the energy of the San Fernando shock. The release of this energy could be over a region 100 times larger than that of the 1971 event. Should such an event occur today, the loss of lives could be in the tens of thousands and property losses in the tens of billions. Neither past earthquake activity nor the possibility of future shocks is limited to western United States. Some eastern and midwestern cities are in areas susceptible to destructive earthquakes. In the interest of public safety and welfare it is important that those concerned take immediate measures to reduce potential earthquake hazards and provide for levels of risk that are acceptable to the general public.

Acknowledgments.—Many persons in government, academic institutions, and private professional practice contributed their time and effort to these studies. The names of many individuals appear as authors of papers. Special recognition and credit must be given to Donald F. Moran who organized much of the engineering effort for the Earthquake Engineering Research Institute (EERI), and also edited many of the papers for engineering content.

Members of the EERI deserve special mention. After the San Fernando earthquake the authors of this paper implemented immediately a preearthquake contract arrangement between NOAA and EERI. The effort by EERI substantially exceeded the terms of the contract, and many EERI members donated much of their time to the field investigations and studies.

The State Role in Seismic Safety

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INTRODUCTION

The lesson of the San Fernando earthquake for public officials in California is clear. Unless the State and local governments act quickly and decisively to formulate a public policy to minimize loss of life and destruction of property by earthquakes, we face a monumental disaster sometime in the future—perhaps tomorrow. As California becomes increasingly urbanized, the danger grows, not only because of increasing population density, but also because urban growth continues to spread into areas where an earthquake once would have caused little damage. And as this danger increases, we must vigorously oppose the idea that the consequences of an earthquake in an urban area are inescapable, that there is little we can do to reduce the risk of death and destruction from a great earthquake.

Yet public officials have only begun to grasp these facts, to accept responsibility for seeing that all reasonable precautions are taken to minimize the threat of a future disaster. This is, of course, no small task. For State officials, there are at least three principal obstacles to effective planning for disaster minimization. First, there is the fact that such planning at the State level must involve a large number of agencies—the Public Utilities Commission, the Department of Water Resources, the Division of Industrial Safety in the Department of Industrial Relations, and the Department of Education, to name a few—on a continuing basis. Thus, there is a real problem of deciding where ultimate responsibility must lie and how maximum participation by all State agencies may be assured. Second, there is the fact that in California, planning has been the jealously guarded prerogative of the cities and counties, and any practical system of disaster prevention must depend heavily on voluntary local implementation of statewide policy. Finally, there is the not unfamiliar reluctance of legislators and administrative officials to

provide the leadership necessary to overcome the other two obstacles in the absence of a major crisis and the resulting public demand for action.

Although State officials in Sacramento were unable to feel the violent shaking of the earth that occurred 350 miles to the south on February 9, 1971, they did feel the subsequent shock wave of public alarm and anxiety. In the past, this might only have resulted in a rash of ill-considered legislation which would have done more harm than good. Fortunately, however, the California Legislature had agreed 2 years earlier to set up a special Joint Committee on Seismic Safety. When the San Fernando earthquake hit, this committee had already begun to study the whole question of seismic safety and risk reduction, not simply in terms of the destruction wrought by a single earthquake, but with regard to the danger facing the State as a whole and with the benefit of knowledge about earthquakes that scientists and engineers have gained over the years. Thus, the committee was prepared to take up the challenge of the San Fernando earthquake in a responsible and thoroughgoing way.

In the 3 years since it was formed, the Joint Committee on Seismic Safety has been attempting to accomplish six basic objectives: to assemble the most advanced current knowledge about earthquakes and associated hazards from the sciences and the engineering professions; to evaluate existing disaster plans throughout the State against the possibility of a great earthquake; to develop more rapid, equitable, and planned programs for recovery and redevelopment after an earthquake; to examine ways in which local government may be better organized and prepared to respond to seismic disaster; to determine the proper role of State agencies and officials in reducing earthquake risk; and, finally, to carefully analyze the proper uses of land in areas of high earthquake risk and the safety precautions that must be put into effect to minimize the loss of life and destruction of property caused by an earthquake. To date, the committee, which has been aided by an outstanding body of advisors drawn from all the diverse fields concerned with seismic safety, has been successful in sponsoring several important pieces of legislation in the California Legislature and in preparing a series of reports dealing with the basic issues of seismic safety.

Its work is far from complete, however. Not only are there several additional pieces of legislation to be brought before the Legislature, but the committee also must find a way to translate its ideas as to the roles of State and local government in seismic safety into concrete plans of administrative action. In doing so it will have to resolve some very difficult questions as to the relationship between State and local government, the desirability of single versus multiple functional responsibility for public safety, and the ability of public officials to maintain an interest in seismic safety between earthquake occurrences.

As we have carried out our work, our guiding belief has been this: In spite of the present unpredictability of earthquakes in terms of time, place, and effect, much can be done to minimize the risk by developing, out of our present knowledge, a set of relatively simple precautionary measures. We know that it is impossible to "earthquake-proof" our urban and rural communities, but we do not believe that that relieves us of all responsibility. Significant reductions in the effects of potentially disastrous earthquakes can be accomplished, we are convinced, and without imposing extreme controls on the development and functioning of our communities.

DEVELOPMENT OF JOINT COMMITTEE ON SEISMIC SAFETY

Joint Committee Formed

In 1968 the Institute of Governmental Studies at the University of California, Berkeley, published a monograph [1] on the public aspects of the earthquake hazard of the San Francisco Bay area. Emphasizing the fact that the urban centers of the Bay area sit astride two major faults and are, therefore, in danger of a violent earthquake at any time, Steinbrugge called attention to the absence of effective public planning for disaster prevention to minimize losses of life and property and to speed postearthquake recovery. His message was not one of impending doom, but a call to begin to do something concrete to prepare for the inevitable earthquakes to come.

Picking up this theme, Stanley Scott, Assistant Director of the Institute of Governmental Studies, published a report calling for the development of a regional plan or program to reduce earthquake hazards through such means as restricting construction in the immediate vicinity of a fault trace, designing

[1] Steinbrugge, Karl V., *Earthquake Hazard in the San Francisco Bay Area: A Continuing Problem in Public Policy*, Institute of Governmental Studies, University of California, Berkeley, 1968.

utility transmission lines to accommodate large earth movements, restricting building on unstable soil in fault zones, and ensuring that new construction is reasonably earthquake resistant. Scott proposed either the establishment of a single-purpose earthquake commission to coordinate earthquake planning in the Bay area or the assignment of that responsibility to a multipurpose regional government agency. The Institute of Governmental Studies, under the direction of Professor Eugene C. Lee, and with the encouragement of Karl V. Steinbrugge, continued to press this idea among Bay area legislators in early 1969, calling for a Bay area study of earthquake hazards and for ways to minimize them "that would be a model for the rest of the world."

In response to this clear challenge to the Legislature's willingness to react to a crisis before it occurred, I agreed to carry legislation at the 1969 session of the California Legislature to create a model regional seismic safety commission. The bill, Senate Bill (SB) 1207, was drawn up with the assistance of Steinbrugge and Scott, among others. It would have created a 27-member San Francisco Bay Area Seismic Safety Commission, composed of State and local officials and several representatives of the public, to study seismic hazards in the Bay area and to develop plans to minimize the potential disaster from a major earthquake in the area. The commission was to be aided by five special advisory groups, made up of engineers, scientists, and appropriate State officials, who were to advise on engineering considerations and earthquake science, disaster preparedness, postearthquake recovery and redevelopment, land use planning related to seismic hazards, and governmental organization and performance in dealing with natural disasters.

Unfortunately, that bill had a very short life. But the idea was not lost; before the session was over, I introduced Senate Concurrent Resolution (SCR) 128 to create a Joint Committee on Seismic Safety in the Legislature with special advisory groups much like those proposed in the defeated bill. With the help of many interested persons and organizations, this measure was eventually passed and the committee formed, its membership chosen to represent southern California as well as the Bay area to assure maximum participation by the persons most interested in making the committee's work a success.[2]

[2] The current membership of the committee is listed in the appendix to this paper.

Advisory Groups

The committee began meeting in January 1970. Its first task was the selection and appointment of 74 advisory group members. Those selected are among the State's leading experts in the various disciplines related to seismic safety, including such fields as insurance, banking and finance, medicine, and land use planning, as well as engineering geology, structural engineering, architecture, city planning, and local government. As was intended under SCR 128, the advisory group members form a unique interdisciplinary pool of talent drawn from the leading departments of the State's institutions of higher education, professional associations, private research groups, and several agencies of State, Federal, and local government.

SCR 128 spelled out the general functions of each of the five advisory groups in these terms:

> 1. *Advisory Group on Engineering Considerations and Earthquake Sciences.*
> This group shall review available scientific and engineering knowledge relative to the reduction of the risks of damage due to earthquake and related geologic hazards. . . . Its judgment in the fields of engineering and earthquake sciences shall guide all other advisory groups. It shall review the progress reports of all other advisory groups and may recommend such changes, elaborations, or new directions as may appear desirable. It shall provide, insofar as feasible, technical information and opinions on submitted questions to other advisory groups.
>
> 2. *Advisory Group on Disaster Preparedness.*
> This group shall be responsible for reviewing and evaluating the adequacy of existing disaster plans insofar as they relate to the probable consequences of disastrous earthquakes. The group should give special emphasis to the development of plans to marshal human, physical, and economic resources necessary to minimize human and material losses flowing from earthquake disaster, and to facilitate restoration of the normal life of the region as expeditiously as possible. . . .
>
> 3. *Advisory Group on Postearthquake Recovery and Redevelopment.*
> This group shall be responsible for recommending a series of general contingency plans to guide the long-term work of recovery, reconstruction, relocation where desirable, and redevelopment. The plans shall include variable parameters of earthquake location, duration, intensity, and damage.
>
> 4. *Advisory Group on Land Use Planning.*
> This group shall determine the limitations that should be placed upon the use of land subject to seismic hazard for appropriate inclusion in the land use plans of State, regional, and local governments.

5. *Advisory Group on Governmental Organization and Performance.*
This group shall study local government organization so as to determine how the plans formulated by the other advisory groups to reduce the risk of loss due to earthquake disaster may best be carried into effect. It shall recommend modifications in existing government organization so that local government jurisdiction is sufficient to enable it to exercise the requisite authority to make the emergency measures effective. It shall recommend new governmental institutions should they appear essential.

For reasons which had nothing to do with the problems of seismic safety or the nature of the tasks assigned the Joint Committee on Seismic Safety, the committee initially was given only $5,000 to carry out its responsibilities. Thus, it was evident that members of the advisory groups would be required to donate their very valuable time in the service of the public interest, which, indeed, they have done with remarkable generosity. Although the funding allocation was increased to $15,000 by mid-1970 (the first year of the study), it was still necessary for the advisory group members to provide even their travel and other overhead expenses out of their own pockets.

San Fernando Study

Then came a new perspective, the result of the violent tremors which struck San Fernando on February 9, 1971. In late April, the California Legislature adopted Joint Rules Committee Resolution No. 7 directing me, as chairman of the Joint Committee on Seismic Safety, to appoint a special three-man subcommittee to conduct "an in-depth investigation" of the "events prior to, during, and after" the San Fernando earthquake. The principal objectives of the study were to learn as much as possible about the effects of the earthquake, giving particular attention to the performance of governmental and private organizations responsible for seismic safety in the area, and to the adequacy of public policies and regulations governing all aspects of seismic safety. The committee was given $150,000 to carry out the study, which was to be completed in 18 months.

Although the committee's assignment was given a new direction, the structure and functions of the existing advisory groups were not changed, nor was the method of obtaining administrative staff services under contract with Diridon Research Corporation, the firm which had assisted with the development of the Joint Committee's organization in 1970.

For the San Fernando study, a special technical team was assembled under the chairmanship of Karl V. Steinbrugge who was given overall technical and scientific responsibility for the project. The other members of the team were: George O. Gates who served as the project's technical coordinator and was responsible for planning and executing the study under Steinbrugge's general direction; Carl B. Johnson, associate technical coordinator and consultant in structural engineering; and eight other consultants, expert in general geologic hazards, dams and soils, hazards to utilities and communications, disaster preparedness, public administration policies related to seismic safety, and land use planning.[3] In addition, a 3-day conference was planned for September 1971 to bring together committee members, the advisory group members, the special consultants and staff, together with certain other selected experts, to deal directly with the problem of determining what is an "acceptable level" of seismic risk.

Earthquake Risk Conference

One of the accomplishments of the Joint Committee on Seismic Safety which, it seems to me, may yield the greatest return over the next few years was sponsorship of this Conference on Earthquake Risk, which was held in Carmel, Calif., September 22 to 24, 1971. The basic question to which the conferees addressed themselves was: How much risk should be tolerated as a matter of public policy?

The question is much simpler in its asking than in its answering. To begin to find an answer, it is first necessary to answer such questions as: How are we to define risk? Is risk a static or a dynamic concept? Who is to be the final arbiter of risk? Is there an acceptable cost/benefit formulation which will help us to establish what ought to be acceptable risk? Quite obviously, a policy of risk reduction must be based upon a systematic exploration of the opportunities for reducing risk—the risk of death and injury to humans and the risk of destruction of property—and the economic, social, and political costs of doing so. We know that as long as we do nothing, the risk to people and property is tremendous; the San Fernando earthquake reminded us of that if we had forgotten. But we need systematic analysis of potential risk, as well as of the potential

[3] Consultants to the special subcommittee are listed in the appendix.

costs of risk prevention, before we can go very far toward establishing an "acceptable risk" goal.

The report of the conference proceedings, which the committee has published, provides much useful material on the dimensions of risk—for example, the psychological, social, and political elements as well as the physical and economic elements.[4] It also provides some very stimulating analyses of risk measurement and the problem of stating risk objectives in terms that will not only satisfy the experts, but the politicians and the general public as well. It was obvious that all of the participants keenly felt the need for more data of many kinds—geologic and seismic especially—but most were also convinced of the need to work with what is available now, rather than postponing action any longer.

More than one participant emphasized the point which must be communicated to the people of California and public officials at all levels: California residents have been extraordinarily lucky with respect to where and when earthquakes have occurred —lucky that they have not occurred at times and places when people are most vulnerable or when the effects might have been reinforced by adverse weather patterns. All of the participants agreed, I believe, that the earthquake risk to life and property in California is now intolerably high, and that it is not only possible but imperative that we take direct and rapid steps toward reducing that risk, not to zero, of course, but to that level which an informed public can accept as reasonable.

Subcommittee Report

The Joint Committee's Special Subcommittee to Study the San Fernando Earthquake published its second report in July 1972.[5] This report consists of a series of seven reports by consultants to the subcommittee covering these topics: the general geological and seismological lessons learned from that earthquake; its impact on dams, soils, buildings, and utilities; certain findings regarding land use planning and governmental organization and performance; and the character and effectiveness of disaster preparedness by State and local agencies. Each of these reports presents a series of recommendations which deserve the close attention of State, Federal, and local policy makers and administrators. The whole list is much too long to include here, but I think it may be useful to list several items that seem to me to be particularly important. These are:

1 At the present time we must assume that seismic risk from ground shaking is relatively high and relatively uniform over large parts of California.

2 Where potential lines of fault rupture can be recognized and accurately delineated, State legislation is necessary to prevent the construction of buildings for human occupancy across the most dangerous of such lines.

3 The State Division of Mines and Geology should be funded to embark immediately upon a systematic program of exploration of selected potentially active faults throughout the State, but particularly in populated areas and areas facing imminent development.

4 The Division of Safety of Dams of the State Department of Water Resources should initiate an immediate review of the seismic stability of all dams having scanty or questionable abutments, all hydraulic fill dams, the danger of valley distortions for all concrete dams, and, as soon as possible, all other earth dams.

5 A permanent State Board on Dam Safety should be established with a continuing responsibility for advising on the adequacy of dam safety standards, the staffing of the Division of Safety of Dams, the enforcement of safety standards, and technical problems related to the safety of dams in California.

6 The State of California should finance a revision of the Recommended Lateral Force Requirements by the Structural Engineers Association of California, incorporating the lessons learned from the San Fernando earthquake and the other seismic design data which have accumulated since the last edition.

7 Minimum statewide seismic code standards should be established, without preempting local jurisdictions having higher building code standards.

8 Legislation should be enacted to require that hospitals and other emergency structures, which are critically needed during a catastrophe, be designed with higher levels of earthquake resistance and damage control than other buildings.

9 Subdivision regulations should be tightened by requiring preliminary geologic reports by registered geologists certified in engineering geology;

[4] *Earthquake Risk*, Interim Report of the Special Subcommittee on the San Fernando Earthquake Study, Conference Proceedings, September 22-24, 1971, Joint Committee on Seismic Safety.

[5] *The San Fernando Earthquake of February 9, 1971, and Public Policy*, Special Subcommittee of the Joint Committee on Seismic Safety, California Legislature, July 1972, Sacramento, Calif.

denying issuance of a public report for any subdivision that fails to take into account hazardous geologic conditions; and requiring that all information included in the original subdivision report be made known to all subsequent lot buyers.

These are only a few of the many general and specific recommendations advanced by the committee's consultants, several of which have already been enacted into legislation. I believe they indicate the thinking which has come out of our study of the San Fernando earthquake and the size of the problem which confronts all of us.

Committee Schedule

The present work schedule of the Joint Committee on Seismic Safety calls for completion of the major work on the advisory groups by June 30, 1973. By that time, it is expected that they will have completed their data-gathering studies and investigations and formulation of recommendations in their several fields of responsibility. Each advisory group will also have had an opportunity to review the tentative recommendations of the other advisory groups and to benefit from public hearings on those recommendations to be held by the committee itself. Finally, work will have begun on the drafting of policy proposals to carry out the advisory groups' recommendations and on the drafting of the committee's final report.

During the following year, from July 1973 through June 1974, the final set of recommendations is to be drawn up and submitted, along with the committee's report, to the Legislature. Bills will be drafted to carry out those recommendations which call for additions to or changes in State law, and members of the advisory groups will be called upon to testify before legislative committees in support of proposed legislation in their fields of expertise.

LEGISLATIVE ACTION

In the aftermath of the San Fernando earthquake, while the pictures of damage to freeways and hospitals and other buildings were still fresh in the minds of the legislators, it was possible to secure quick passage of several significant pieces of legislation. The most important of these was a bill establishing a State strong-motion instrumentation program. The program is being organized and monitored by the State Division of Mines and Geology, with the assistance of an advisory board appointed by the State Geologist and including a representative of the Earthquake Engineering Research Institute, the National Oceanic and Atmospheric Administration, the U.S. Geological Survey, the Structural Engineers Association of California, the Earthquake Engineering Research Center at the University of California, Berkeley, and the Earthquake Engineering Research Laboratory at the California Institute of Technology. The legislation authorizes the State Division of Mines and Geology to purchase and install strong-motion instruments "in representative structures and geologic environments throughout the State as deemed necessary and desirable by the advisory board." The Division also is authorized to contract with the National Oceanic and Atmospheric Administration ("or other competent agencies") for the maintenance and service of the instruments and with appropriate State agencies for the collection, interpretation, and publication of all records from the instruments. To pay for the program, all counties and cities of the State are required to collect a fee from building permit applicants equal to 0.007 percent of the total valuation of the proposed building. Cities and counties may be exempted from this requirement only if they have already adopted ordinances requiring the installation of accelerographs in specified structures.

In 1971, the Legislature also enacted a State School Building Aid and Earthquake Reconstruction and Replacement Bond Law authorizing, when approved by the State's voters, a $350 million bond issue, from which up to $250 million is to be made available to local school districts for the purpose of rehabilitating, reconstructing, or replacing structurally hazardous school facilities constructed prior to enactment of the 1933 Field Act—earthquake safety legislation that resulted from the destruction of school buildings in the Long Beach earthquake of that year. The bond issue was subsequently approved by the voters in June 1972, and the funds are now being allocated to those districts that have a demonstrated need.

The Joint Committee on Seismic Safety was also successful in sponsoring legislation to require cities and counties to adopt seismic safety elements as a part of their general plans. State law requires California's counties, noncharter cities, and (since 1971) charter cities to adopt a "comprehensive, long-term general plan for the physical development of the county or city . . ." containing elements prescribed

by State law. The committee's bill added to existing requirements a "seismic safety element consisting of an identification and appraisal of seismic hazards such as susceptibility to surface ruptures from faulting, to ground shaking, to ground failures, or to effects of seismically induced waves such as tsunamis and seiches." The requirement was purposely kept general so as to establish the concept without attempting to lay down a definitive requirement before it was possible to do so. The Council on Intergovernmental Relations is now developing a more detailed statement with a draft of a model seismic safety element for use by local planning agencies.

A related piece of legislation, passed in 1971, states that local zoning regulations must be in conformity with adopted city and county general plans by January 1, 1973. Unfortunately, charter cities—and all of California's major cities are charter cities—are excluded from this requirement at present.

Two unsuccessful companion measures, SB 298 and AB 1176, dealt with subdivisions. Among other things, they would have required: preparation of a preliminary geologic report by a registered geologist certified in engineering geology; withholding of a public report on a subdivision (required before lots can be sold) by the real estate commissioner if the subdivision failed to take into account hazardous geologic conditions; and adoption of local ordinances requiring preliminary geologic reports for all subdivisions according to standards to be adopted by the Commission on Housing and Community Development. A related bill, also unsuccessful when first introduced, required a geologic report to be given to every purchaser of a new subdivision lot or parcel and a copy of the report to be attached to the property for the benefit of all subsequent buyers.

The fact that a number of these bills were defeated was disappointing, but it does not mean that the effort to seek such legislation should be abandoned. It is not at all unusual for significant legislation to take 2 to 3 years for passage. Consequently, several of the bills which were defeated in 1971 were reintroduced in 1972 in somewhat modified form.

1972 Legislation

One of the most important pieces of legislation in the field of seismic safety introduced at the 1972 session of the California Legislature was a bill to ensure that hospitals be constructed with the same attention to earthquake resistance as is required in school construction under the 1933 Field Act. As was noted in the preamble to the bill, the San Fernando earthquake, although relatively moderate in terms of total energy release, resulted in the collapse of or extensive damage to a number of hospitals, rendering them all but inoperative. Several of the most heavily damaged hospitals were of recent construction and had been designed and built to meet construction standards prescribed by local government.

The enactment and enforcement of building codes have always been a local prerogative, yet the San Fernando earthquake demonstrated all too tragically the failure of local government to adopt and enforce effective earthquake-resistance standards for hospital construction, just as the 1933 Long Beach earthquake demonstrated the folly of leaving public school construction standards to local agencies. Hospitals, like schools, house large numbers of persons who cannot always be evacuated at the moment an earthquake tremor is felt. Moreover, hospitals located in the vicinity of a major earthquake must be available for the emergency care of the victims of the earthquake.

Thus, the bill which I introduced, SB 519, directed the State Department of Public Health to contract with the Office of Architecture and Construction in the Department of General Services to review and approve plans for the construction or alteration of all hospitals, to insure that the structures will provide reasonable protection against the potential effects of earthquakes. All hospital construction plans are to be required to include an assessment of the nature of the site and the potential for earthquake or earthquake-related damage, based upon an engineering geologic investigation. In addition, the Department of Public Health is to be authorized to perform, on request from any existing hospital, an examination and report on the structural condition of the hospital. Finally, the bill creates within the department a Building Safety Board, made up of 17 members, including such officials as the State Fire Marshal, the State Geologist, the State Architect, and the chief structural engineer in the Schoolhouse Section of the Office of Architecture and Construction, to act as an appeals board "in all matters affecting

seismic structural safety" under the new law. The bill was enacted into law as chapter 1130.

Another important piece of legislation also passed this year is a measure to require the State Geologist to delineate "special study zones" encompassing "all potentially and recently" active traces of the State's major faults. Within such zones, ordinarily ¼ mile or less in width, the city or county having jurisdiction is to be required to approve every proposed new real estate development or structure for human occupancy in accordance with policies and criteria to be established by the State Mining and Geology Board, which is to be expanded to include at least one seismologist, one architect, one soils engineer, and one geologist with knowledge of soils behavior. A bill specifying that the seismic safety element in city and county general plans shall also include an appraisal of mudslides, landslides, and slope stability as geologic hazards has been passed by the Legislature and signed into law.

Another successful measure requires all owners of dams (local agencies, utilities, etc.) to file inundation maps with the State Office of Emergency Services, showing the areas of potential flooding in the event of dam failure. Cities and counties having jurisdiction over areas where death or personal injury may occur because of dam failure are required to adopt emergency evacuation plans to be reviewed by the Office of Emergency Services. These emergency plans are to include a delineation of the area to be evacuated, routes and traffic control measures to be used, shelters to be activated, methods of transportation, special procedures to evacuate persons from "unique institutions," protection of the evacuated area, and procedures for reentry.

The Legislature also agreed to earmark $30 million in new tax revenues for a School Building Safety Fund to aid school districts in strengthening or replacing school buildings which do not meet the State's school construction standards for earthquake resistance. A part of these funds may be used to supplement regular State school construction loans and a part for grants to the most hardpressed districts. A related measure, which was approved by the voters in November 1972, amended the State's constitution to lower the required vote in school bond elections from two-thirds to a simple majority when the funds are to be used for repairing or replacing structurally unsafe buildings.

Disaster Aid

Another fact brought out by the San Fernando earthquake is the need to protect innocent victims of earthquake damage from the financial losses they might otherwise suffer.

At present, relief is provided principally in the form of Federal Government loans and grants to disaster victims. Such aid is fully justified when it is carefully administered, but, unfortunately, one consequence of the Federal Government's generosity in providing low-interest loans and outright grants to the victims of disasters is to soften the resolve of those who live and work in the area of the disaster to see that the risk of future disasters is reduced. Following the San Fernando earthquake, Congress granted residents of the area who sustained damage to their homes emergency loans which not only carried a 5.8-percent interest rate (recently reduced to 3 percent), but also had a forgiveness feature for the first $2,500 that made that much of the loan an outright grant. And now the Federal Government has further liberalized such emergency loans, increasing the grant portion to $5,000 and reducing the interest on the balance to 1 percent. The general concept is certainly laudable, but I think we must give more careful thought to the way it is carried out. The forgiveness feature of these "loans" not only invites fraud, as has occurred in the San Fernando case, but adds to the incentive to simply rebuild, replace, or expand damaged structures with no concern for the fact that an earthquake may strike again with the same or even more severe effect, and the population will be no better protected.

It seems to me that the agency which administers such postdisaster aid must at least cooperate with whatever agency or agencies are charged with reducing the risk of future disasters. In some cases, for example, it may be much wiser to pay for the relocation of residences and businesses that have been located in a high risk area (in a path of slides or across a faultline, for example) than to immediately reconstruct them in the same spot. At the very minimum, the agency which is to pay for reconstruction should be given a preliminary assessment of the risk of a repeat of the disaster and recommendations for reducing that risk.

It is also evident that a way must be found to provide earthquake insurance to businesses and homeowners at a reasonable cost. A complete system

of catastrophic compensation is needed to provide financial protection against earthquake damage, floods, landslides, and seismic sea waves. The State must find a way to permit and encourage the insurance industry to expand its activity in this field so that earthquake insurance is brought within the reach of all.

State Capitol

The renewed interest in seismic safety resulting from the San Fernando earthquake has not only affected the deliberations within the State Capitol in Sacramento, but has involved the century-old building itself. Two recent studies undertaken at the request of the Joint Committee on Seismic Safety have reported that an earthquake of the magnitude of the San Fernando earthquake might very well cause the capitol dome and one or both of the legislative chambers to collapse, resulting in injuries and deaths not only to legislators and other State officials but also to many school children and other visitors who throng through the old building in the daytime. According to the State Architect, the building is constructed mostly of unreinforced brick, which was adequate for the original design but which, with the addition of a fourth floor and many other alterations over the years, has become increasingly vulnerable and hazardous.[6]

It is estimated that it will cost some $41 million to make the old building reasonably safe (a more recent annex housing the Governor's office and many legislative offices and hearing rooms is believed to be safe now), if the State chooses to do so. Otherwise, it is clear that the legislative chambers and the offices on upper floors should be abandoned and tourist traffic sharply curtailed. There is little evidence, however, that the majority of the State's legislators is willing either to spend the money to strengthen the building or to abandon it, despite the clear evidence of danger. The only action the Legislature has taken to date has been to ban guided tours for school children through the older part of the building, including the legislative chambers and the offices of the Secretary of State, Lieutenant Governor, and State Treasurer.

Other State Action

The unprecedented ground shaking that occurred during the San Fernando earthquake took a heavy toll of freeways in the area, destroying an estimated $15 million in freeway facilities. These freeways had been designed and constructed according to the lateral force requirements recommended by the Seismic Committee of the Structural Engineers Association of California, based upon seismograph readings taken from the El Centro earthquake of 1940, but these standards proved inadequate, particularly for bridges and overpasses. Had the earthquake occurred during the morning or evening rush hour, there undoubtedly would have been considerable loss of life among motorists.

From the information gained from the San Fernando earthquake, the State Department of Public Works has adopted new design standards, and a $5 million program is now under way to improve the earthquake resistance of several hundred State highway bridges that do not meet the new standards. The principal modification has been the installation of restrainers on bridge expansion joints to prevent the connected spans from separating entirely when severely shaken. To see that this work is carried to completion, the Legislature adopted a resolution this year directing the department to submit to it biannual progress reports on "earthquake-proofing" design advances in highway and bridge construction, including remedial action planned, under way, and completed by the State for existing State highways.

NEXT STEPS

Public policy in earthquake safety, as in nearly every other field, as Stanley Scott has emphasized, must be based upon a partnership that includes Federal, State, and local government.[7] Each of the three has important contributions to make, but each also has limitations that prohibit it from doing the whole job itself. The Federal Government commands a large share of the necessary resources (we are perhaps too often inclined to think) and is relatively free from vulnerability to day-to-day political pressures that so often separate good intentions from actual accomplishments. It may also have better access to the scientific and technical resources essential to effective action in this field, but I am inclined to think not. A state of the size of California with its vast reservoir of education, research, and industry

[6] Seismic Study, West Wing, California State Capitol; California Office of Architecture and Construction, June 1972, pp. 21–26.

[7] Toward a Partnership for Seismic Safety: 1974 and Beyond, Stanley Scott, Institute of Governmental Studies, University of California, Berkeley, April 26, 1972.

should be able to mobilize the necessary scientific and technical resources without having to depend upon Federal agencies, except in very special circumstances.

Local government generally lacks the financial resources and the breadth of responsibility to deal effectively, unaided, with the problems of seismic safety. Moreover, local government is often all too vulnerable to the kind of pressures that block or erode any public action which necessarily restricts the freedom of private economic interests. This is not always true, of course; in many instances local government can act quickly and decisively to deal with conditions that are much too localized and specific for effective action at the State or Federal level. Yet even when willing to act decisively, local government frequently lacks the human and financial resources to accomplish its objectives. This fact was brought out quite clearly by the San Fernando earthquake. As has been pointed out elsewhere, local government is often "too big to do the little things and too little to do the big things."

State government, on the other hand, is big enough to do the big things, or most of them, if it has the will and the popular backing. The essential problem is to impress upon legislators and other elected officials, who are daily confronted with a host of major problems requiring their attention, the importance of acting to minimize the consequences of an earthquake or other natural disaster before the disaster occurs. Unfortunately, it is a much easier matter to get State officials to respond to a disaster which has just occurred, to provide such financial or other aid as the victims may need, than to get them to take those steps necessary to reduce the danger to people and property from future acts of nature. This is true not because State officials are by nature unimaginative and slothful, but because they seldom have the broad popular support for preventive action—action which must always arouse some opposition from affected interests—that they have for disaster relief. Thus, those who want State governments to play an important role must prepare their case carefully and be prepared to do what is necessary to generate public support.

There are, I believe, five principal functions that State government can perform in the interests of advancing seismic safety:

1 *Policy Objectives.* It is up to State government to establish the basic policy objectives for seismic safety within State boundaries—to define as precisely as possible what is to be the minimum acceptable level of risk to life and property and the principal factors to be taken into account to reduce risk to that level. This is not, of course, a one-time job, for the definition of acceptable risk will necessarily change as do the elements of such a definition—technical and scientific knowledge, economic and social costs, political leadership, and the popular sense of urgency. Nor is it one that is easily accomplished; it is at this first stage that we must exercise the political leadership necessary to overcome the pervasive social lethargy that, except in time of great emergency, so often blocks essential public action.

Dr. John A. Blume, in a paper presented at the Joint Committee's Earthquake Risk Conference in 1971, suggested that a goal of 10 to 12 or fewer fatalities per person-hour of exposure would be realistic "on a long-term improvement basis." This is the sort of concrete recommendation that politicians need from the scientific and technical experts—not to enact into law as a once-and-for-all solution to the problem, but to serve as a guideline for assessing administrative progress by State and local agencies in planning for the minimization of earthquake disaster. Of course, we must go about establishing such objectives with some care so that we do not appear to be trying to frighten people into an extreme reaction or, conversely, that we seem to be acting with callous disregard for those whose lives and property fall within the boundaries of acceptable risk. In this matter we can, perhaps, learn something from those Federal agencies that have struggled with similar problems in the fields of food and drug administration, nuclear radiation safety, and environmental protection.

2 *Statewide Planning.* In California, we have so far delegated most of the responsibility for public planning to local government, and this, of course, includes land use planning related to seismic hazard reduction, as well as environmental protection and other similar functions. In some areas, this has worked well; in fact, all of our examples of truly effective land use planning and administration are to be found at the local level. Yet too often local planning officials and their elected employers are unable to contend with opposing interests because they are unable to see their efforts in a broader context. Moreover, planning at the local level in California remains highly fragmented, with responsibility scattered among a multitude of individual agencies

that are wholly preoccupied with their own concerns. If this is to be changed, the State must establish an effective planning agency of its own which can provide local planners and administrators with a statewide view of the problems—seismic and otherwise—with which they must cope at the local level. The State also must be prepared to play a direct part in the resolution of some of the toughest planning problems, such as those relating to powerplant siting, coastal development, and enforcement of land use controls in the highest risk areas of the State, which experience has shown are beyond the powers of local government.

3 *Resource Collection and Allocation.* The State is also the proper agency to take responsibility for marshaling the basic resources necessary to carry out a policy of seismic risk reduction. By basic resources, I mean not only tax revenues, but also the human capital and the knowledge which must be combined to deal with the problem. The State can be of immeasurable aid to local government by collecting and disseminating current knowledge concerning earthquake safety and risk reduction, and by encouraging the expansion of that knowledge by the State's public institutions of higher education, as well as by gathering and focusing State and Federal financial resources for allocation where they are needed most.

The work of the Joint Committee on Seismic Safety demonstrates the value of this approach. Prior to the San Fernando earthquake and formation of the Committee, there was remarkably little continuing concern on the part of the public, the Legislature, or the administration about earthquake safety. I believe, however, that the committee and its advisory groups have begun to be a major force for change by bringing together the State's leading geologists, structural engineers, urban planners, and others to pool their knowledge of what can and must be done to minimize the potential damage and loss of life from an earthquake, and by creating a means for transmitting that knowledge to the public officials at all levels who must take action to put that knowledge to work.

4 *Technical Assistance.* A very closely related function of State government in this area should be to provide technical assistance to local government to help local officials implement statewide planning at the community level. Technical assistance includes such things as developing model building codes, model zoning regulations, geologic mapping, assistance with public education programs, and disaster relief organizations plans. Any workable statewide plan must make provision for extending such assistance on a regular basis.

5 *Public Information.* All too obviously, efforts to reduce the likelihood of severe damage to property and loss of life will fail unless we find a way to establish and maintain a minimum public awareness of earthquake risk and of the possibility of minimizing that risk, before disaster strikes, through well-planned public action. In the absence of such an awareness, we can be assured that neither State nor local officials will be able to accomplish much in the way of increasing seismic safety.

We should not delude ourselves by thinking, for example, that it is an easy thing to translate technical knowledge as to the location and general character of various earthquake hazards into effective land use controls. It is still an open question as to how much authority local and State government should and can exercise over land use because of geological hazards. Such controls affect not only large land developers, but also many individual small property owners and many tax-supported entities. Small, as well as large, landowners will be unhappy to find that they cannot use the land they own for the purpose for which they obtained it, or, even if there is no direct prohibition, that financial institutions are unwilling to lend them construction capital for sites which have been identified as hazardous. Effective land use planning, like many of the other elements of a comprehensive effort to reduce earthquake hazards, depends heavily upon broad public acceptance, and that acceptance will come only from a basic public awareness of seismic hazards and the steps which must be taken to minimize their impact on life and property.

Therefore, one of the most important things the State can do is to foster this public awareness—not by issuing reports that are simply designed to frighten without informing—but by bringing earthquake hazards to the attention of the citizens of the State in an undramatic, yet forceful, way and then insuring that public awareness is not eroded away over the period when the earth is still.

Seismic Safety Commission

"Legislative actions have been, and will continue to be, successful in providing the framework and authority for executive action on many fronts at different levels of government, towards solution of

the earthquake problem" Karl Steinbrugge has observed; but, as he has gone on to say, "a unifying influence and authority" is also needed, "owing to the complexity of the problem and the complexity of executive action needed to forestall future inconsistencies in the application of legislative actions and the possibility of ignoring developing requirements in earthquake hazard reduction." [8] I agree entirely. The present work of the Legislature through the Joint Committee on Seismic Safety may come to little a few years from now if we do not move to establish a permanent body to oversee the implementation of recommendations arising out of our study.

There are at least five principal tasks for such an agency or commission, none of which the Legislature or any existing agency is particularly well suited to perform. These are: to prepare a statewide plan for seismic safety to reduce the risk of death and destruction to an acceptable level throughout the State; to coordinate and monitor the seismic safety activities of existing State agencies; to support and encourage effective seismic safety efforts at the local level in conformance with the statewide plan; to keep the public informed as to the current level of risk statewide, and in each principal community, to lives and property; and to encourage scientific and technical efforts to expand knowledge of earthquakes and the requirements of seismic safety. In addition, if experience proves it necessary, the new body might be given power to enforce safety standards directly where the nature of the hazard or the failure of other agencies leaves no alternative.

It is too soon, I believe, to spell out in detail the organization and character of such a body. However, it clearly should be one which is not dominated by experts who might be accused of promoting their own interests rather than the public interest, and it should be one which is responsible to the Legislature as well as to the executive branch of State government. Ideally, it will be able to communicate directly with all State agencies which have a significant interest in seismic safety and it will have the stature to ensure that all State agencies follow whatever guidelines are established in the statewide seismic safety plan. Finally, it must be a body which has sufficient authority and responsibility to maintain the interest of those who are appointed to it, so that they do not themselves become the victims of the very complacency which they are to help overcome.

The following specific tasks might be accomplished immediately by a statewide body such as I have suggested. These tasks have been recommended by one or more of the official groups which have studied the problem, including the Task Force on Earthquake Hazard Reduction of the President's Office of Science and Technology and the panels of consultants assisting in the work of the Joint Committee on Seismic Safety.

1 To channel funds to counties, cities, and smaller communities to enable them to conduct surveys of building types that are most hazardous and are located in the highest risk areas. The information gained from such surveys should be useful not only to inform the communities as to the existing levels of risk, but also to make it possible for local officials to make better informed judgments concerning the costs of risk reduction.

2 To sponsor demonstration projects in the application of cost/benefit analysis to earthquake risk reduction in selected communities representing a wide range of probable risk. It has proven much easier to talk about the necessity for cost/benefit analysis as a tool for public policy formulation than to actually apply such analysis to concrete situations —regarding earthquake risk reduction as well as all other elements of land use planning. It is time to relate theory to reality, to find out what we need to know about the costs and benefits of cost/benefit analysis in this field. The analysis should be as comprehensive as resources and imagination permit; it should include such factors as the cost of required technical services for property owners, the cost of condemnation and clearance projects where necessary, and the full cost of postdisaster recovery and rehabilitation.

3 To conduct a survey of all State buildings that house large numbers of State employees or the public, or that may serve as rallying points in time of earthquake danger, as to their earthquake resistance. The recent reports on the vulnerability of California's Capitol Building underline the importance of State officials practicing what they preach to local government.

4 To sponsor a detailed study of State and local taxes to determine the extent to which they encourage construction and land use patterns which run

[8] Karl Steinbrugge and George O. Gates, "A Unifying Objective" in *The San Fernando Earthquake of February 9, 1971 and Public Policy*, Joint Committee on Seismic Safety, July 1972, p. 127.

contrary to seismic safety efforts, and to recommend changes in the tax structure to correct this condition and, if desirable, to provide incentives for construction and land use practices which reduce earthquake danger.

5 To establish a team of experts in several fields —engineering geology, structural engineering, building code enforcement, disaster recovery, etc.—that can be called upon to conduct immediate on-the-scene investigations of minor, as well as major, earthquakes as they occur, and to coordinate similar postearthquake studies by other Federal, State, and local agencies.

These are some of the things that might be started at once. For the longer run, the final report of the Joint Committee on Seismic Safety will have a great deal more to recommend. The committee and the members of the advisory groups have worked diligently from the start to develop a set of findings and recommendations that will be of real value in substantially reducing the threat of seismic catastrophe. I hope the lessons of the San Fernando earthquake will still be sufficiently fresh to enable us to implement those recommendations.

Much has already been accomplished to make California a safer place in which to live and work, but much remains to be done. There is no need for us to continue the past pattern of disaster, alarm, and complacency, repeated over and over. It is the responsibility of public officials, scientists, technicians, and other professional experts to pool their knowledge and experience in a manner that will make it possible for an informed public to break out of that old pattern of behavior. I believe the work of the Joint Committee on Seismic Safety represents a major step in that direction.

APPENDIX: COMMITTEE MEMBERS, ADVISORY GROUP CHAIRMEN, AND CONSULTANTS

Joint Committee on Seismic Safety:

Senator Alfred E. Alquist (Chairman)
Senator Joseph M. Kennick
Senator Alfred H. Song
Senator James E. Whetmore
Assemblyman Jim Keysor (Vice Chairman)
Assemblyman Leroy F. Greene
Assemblyman Richard D. Hayden
Assemblyman Paul Priolo

Special Subcommittee for Study of the San Fernando Earthquake:

Assemblyman James A. Hayes (Chairman)
Senator Joseph M. Kennick
Assemblyman Jim Keysor

Advisory Group Chairmen:

Executive Committee: Karl V. Steinbrugge
Engineering Considerations and Earthquake Sciences: Gordon B. Oakeshott
Governmental Organization and Performance: Marcella Jacobson
Land Use Planning: George G. Mader
Disaster Preparedness: Robert A. Olson
Postearthquake Recovery and Redevelopment: Will H. Perry

Consultants to the Special Subcommittee on the San Fernando Earthquake:

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C. Martin Duke, Professor of Engineering, University of California, Los Angeles.

George O. Gates, Geologist.

Carl B. Johnson, Consulting Structural Engineer, Johnson & Nielsen Associates.

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Donald F. Moran, Consulting Structural Engineer.

Richard W. Roether, Planning Consultant, Eugene D. Wheeler & Associates.

H. Bolton Seed, Professor of Civil Engineering, University of California, Berkeley.

Karl V. Steinbrugge, Structural Engineer, Insurance Services Office, San Francisco.

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Introduction to Study of Earthquake Effects on Buildings

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21 Acknowledgments

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The San Fernando earthquake of February 9, 1971, although moderate in magnitude, provided a real test for many types of building structures. More usable strong-motion accelerograph records were obtained than from all previous earthquakes combined. History has shown that the greatest impetus for improvements in earthquake-resistant design is provided by actual earthquakes. Between earthquake occurrences, progress is dependent on research and studies of past earthquakes. The tendency is for interest to lag and to decrease rapidly following a shock.

Regardless of the amount of research and studies, the earthquake provides the ultimate test. Mistakes in judgment, faulty theory, and poor construction practices are located easily by the earthquake.

Immediately following the earthquake, a cooperative effort was initiated between the Earthquake Engineering Research Institute (EERI) and the Structural Engineers Association of California (SEAOC) to study the behavior of buildings and to prepare reports for publication by the National Oceanic and Atmospheric Administration (NOAA). Members of SEAOC, leaders in the building earthquake engineering field, developed the SEAOC Code which forms the basis for earthquake-resistant provisions in most California and United States codes. C. W. Pinkham, president of Structural Engineers Association of Southern California (SEAOSC) for 1971, served as chairman of the NOAA/EERI Subcommittee on Buildings. A Buildings Editorial and Review Subcommittee guided this phase of the study.

SELECTION OF BUILDINGS

The initial task of the committee involved selection of buildings to be studied. Most of the studies were done by private consultants under subcontract to EERI. The available funds and time schedule required that the number of structures be limited. It

was decided that the major effort should be expended on studies of modern earthquake-resistant structures because they offered the greatest opportunity to determine if present design criteria are adequate.

There were several obvious choices in the earthquake-resistant category, namely the major structures in the heavily shaken area. Most of these structures happened to be medical facilities—Olive View Hospital, Holy Cross Hospital, Indian Hills Medical Center, and Pacoima Memorial Lutheran Hospital. Two major juvenile detention facilities suffered major damage—the San Fernando Juvenile Hall and Camp Karl Holton.

Damage to modern one-story industrial and commercial structures with wood roof systems was common and severe in the area of strong ground motion. A number of these were chosen for study because of their location and type of construction. Partial collapse of this widely used type of construction could have resulted in injury and loss of life if the structures had been occupied at the time of the earthquake. A paper in this volume, "Behavior of Joist Anchors Versus Wood Ledgers," compares degrees of damage and construction details for this type of construction.

Another major category of buildings studied included those tall buildings that contained three strong-motion instruments. These studies, relatively expensive due to the computer costs involved, were performed by consultants under contract to EERI. In order to select buildings for study, a subcommittee on instrumented buildings carefully reviewed all strong-motion records and inspected individual buildings. A list of buildings for study was then made in order of descending priority. Factors affecting the priority included quality of instrument records, building location, structural type, and construction materials. The availability of digitized records was also a consideration. As many buildings as funding would permit were chosen from this priority list for inclusion in this study. Some building reports were donated by firms. These are acknowledged in this volume in the paper "High-Rise Buildings With Strong-Motion Instruments—Dynamic Analyses." One other tall structure, Union Bank on Ventura Boulevard, was chosen for study, but did not contain strong-motion instruments. This building suffered some structural damage.

The behavior of public school facilities was included since they represented both nonearthquake-resistant and earthquake-resistant designs. Also, the design and construction of public schools since 1933 has been checked and supervised by a State agency, placing special emphasis on lateral force-resisting systems.

Some old, unreinforced masonry structures were included to emphasize the potential hazard of these buildings. The greatest loss of life was caused by the collapse of such a building at the Veterans Administration Hospital.

Other important considerations in selecting buildings for study were the availability of design drawings and damage data and cooperation of the owner. The majority of the structures studied, except for several tall buildings near downtown Los Angeles, suffered moderate-to-severe damage, and even collapse. The committee realized that much can be learned from undamaged structures that are subjected to strong earthquake motion, but it was felt that more could be learned from the analyses of damaged structures. Limited funds made it necessary to confine the studies to damaged structures. Nevertheless, building reports mention adjacent undamaged structures when these were present.

Several repair techniques were used following this earthquake and a paper covering this aspect is included.

Studies of additional building structures are included in Volume II in the discussion of energy, communication, water and sewerage, and transportation systems.

Careful consideration was given to the organization and treatment of material on building damage because many readers of the report will not be structural engineers or those practicing earthquake engineering. The paper titled "History and Philosophy of California Earthquake Codes and Elements of Lateral Force Design" explains building codes that are in local use and defines many technical terms that are used in the building reports. The building reports are written for those who possess an engineering degree or equivalent experience; they are not intended for the layman.

The building reports are arranged in two general groups—earthquake-resistant and nonearthquake-resistant buildings. Within the former, the low-rise (mostly single-story) industrial buildings appear together, but are subdivided further into groups depending on construction materials and locations. De-

tention facilities and hospital and medical facilities are separated by use and by occupancy regardless of location or construction materials. This was done because these facilities probably should have special and different design criteria due to their importance and forced-type of occupancy. All instrumented tall buildings are grouped together, but are subdivided according to the consultant involved because slightly different analysis procedures were used by different consultants. Old, unreinforced masonry structures are in one group; the Veterans Administration Hospital is treated separately from this group because it contained both old and new structures. A separate paper covers public school buildings.

Other papers related to buildings treat "Earthquake Damage Repair Techniques," "Earthquake Damage and Related Statistics," "Building Period Measurements," and "Nonstructural Damage."

A map showing ground breakage is included as part of the geology treatment in Volume III. Soils and Foundations follows the treatment of buildings in Volume I, Part B. The Subcommittee on Soils and Foundations prepared case studies at many building sites. These case studies are referenced in building reports as applicable.

When individual building reports are grouped, an introductory statement applies to all building reports in that group. Common conclusions and recommendations are summarized at the end of most groups. Only those conclusions and recommendations that are germane to individual buildings in a group are included within individual building reports. Where an author is not listed for these statements, they are prepared by the Buildings Editorial and Review Subcommittee.

One very important criterion in organizing the volume was the requirement that every report and paper contain a section on conclusions and recommendations as to how performance could have been improved. The committee felt that these would be the most significant aspect of this report. It is hoped that consideration of these conclusions and recommendations will lead to meaningful revisions in earthquake-resistant design criteria.

ACKNOWLEDGMENTS

All material in this volume, except the papers on "Earthquake Damage and Related Statistics" and on "Building Period Measurements," was prepared under the guidance of the NOAA/EERI Subcommittee on Buildings. Members of this subcommittee are identified with their respective papers or building reports. The editorial and review portion of this subcommittee is listed below.

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Lynn W. Bockemohle, structural engineer, Wheeler & Gray, consulting engineers, Los Angeles, Calif., worked as a member of the NOAA/EERI staff throughout the study and was personally responsible for coordinating the committees and work of the consultants who contributed to this volume.

Christopher Rojahn represented NOAA as liaison, helped direct the entire investigative effort, and reviewed and edited papers.

The subcommittee gratefully acknowledges the

contributions of the individual authors and firms who are identified with their respective papers or building reports.

Additional subcommittee members who were active in the preparation of various sections of this report are identified in the section on instrumented buildings and the paper on "Nonstructural Damage."

Karl V. Steinbrugge, professor of structural design, University of California, Berkeley, kindly contributed the paper "Earthquake Damage and Related Statistics."

The paper on building period measurements was prepared under the guidance of a NOAA/EERI subcommittee under the chairmanship of Ralph McLean, partner, McLean & Schultz, civil and structural engineers, Fullerton, Calif.

The various local building departments cooperated by making their damage reports and file sets of drawings available and by reviewing building reports which were prepared by consultants.

The Subcommittee on Soils and Foundations arranged for and obtained copies of all available foundation reports for the buildings being studied.

Special recognition of the efforts of many members of the Structural Engineers Association of California (SEAOC) is acknowledged. The material on buildings could not have been assembled without their cooperation. The volume on buildings is strengthened considerably by the wide range of expertise in analysis and review, which was brought to bear by the cooperation of SEAOC.

History and Philosophy of California Earthquake Codes and Elements of Lateral Force Design

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INTRODUCTION

This paper presents a brief history of earthquake-resistant provisions in California building codes and a short explanation of terms used in lateral force design. No attempt has been made to cover all aspects of this constantly changing subject. The intent is to enable the reader, who is not thoroughly familiar with earthquake engineering, to better understand the individual building discussions that follow. The paper is limited to California codes and practices, many of which are used in other parts of the United States. The list of references is a starting point for those who desire to explore the subject in depth. The authors are indebted to the members of the NOAA/EERI Subcommittee on Buildings for their constructive comments and suggestions.

HISTORY OF CALIFORNIA EARTHQUAKE CODES

Following the 1906 San Francisco earthquake, increased wind loadings were suggested to make tall buildings more resistant to lateral loads. However, the 1923 Tokyo earthquake resulted in the first attempt to codify earthquake-resistant design. A lateral force coefficient of 10 percent of gravity was used in Japan for general structures.

No significant progress was made in California until after the Santa Barbara earthquake of 1925. This shock resulted in the U.S. Congress directing the U.S. Coast and Geodetic Survey—now the National Ocean Survey, a major component of the National Oceanic and Atmospheric Administration (NOAA)—to assume responsibility for investigations and reports on seismology. Between 1925 and 1927, several California communities adopted earthquake code requirements with lateral force coefficients of 20 percent of gravity.

The first edition of the Uniform Building Code (UBC) of the Pacific Coast Building Officials Con-

ference (now known as the International Conference of Building Officials) was published in 1927 and contained in the appendix a chapter on earthquake provisions, planned for optional use. The 1927 UBC lateral force coefficient was 10 percent of gravity.

The Long Beach earthquake of March 10, 1933, provided the impetus for more stringent seismic codes in California. Two States' acts dealing with earthquake-resistant design criteria were passed by May 26, 1933.

One of these, the Field Act, authorized the State Division of Architecture to review and approve all public school plans and specifications and required continuous supervision of the construction. The enforcement of this Act is now performed by the Schoolhouse Section of the Office of Architecture and Construction, Department of General Services.

The second law, the Riley Act, made provisions for design and construction of buildings, other than dwellings and farm buildings, to resist seismic and wind forces. This Act was formulated for more general application than the Field Act, which applied only to public schools. The original Riley Act specified a lateral force design coefficient of 2 percent of the gravity loading, and several modifications have been made from time to time. This Act now requires basically the same level of seismic force as the 1961 UBC.

Earthquake-resistant code provisions were adopted in 1933, or shortly thereafter, by the city and county of Los Angeles, the city of Long Beach, and other municipalities, primarily in southern California. These early provisions were founded largely on Japanese experience and judgment and were based on behavior of buildings in past earthquakes. A 10 percent of gravity base shear coefficient was commonly used.

Some early codes contained provisions for increasing lateral force factors for poor soil conditions, based on allowable design soil pressures. Soil amplification factors have since disappeared from all California codes, except for some requirements for pile-foundation interconnection.

The main reasons for the elimination of a soils amplification factor have been the lack of knowledge as to what the factor or factors should be, and the lack of adequate instrumental records on varying soil types. The February 9, 1971, earthquake resulted in many strong-motion records, but considerable research will be required before meaningful recommendations can be made.

The city of Los Angeles, in 1943, recognized indirectly the influence of flexibility on building response and adopted the formula:

$$C = \frac{60}{N + 4.5}$$

where:

C = Coefficient in percent of dead load; and
N = Number of stories above the story under consideration. (The maximum number of stories was 13.)

In 1957, when the height limit was removed in Los Angeles, this formula was changed to:

$$C = \frac{4.6S}{N + 0.9\ (S\text{–}8)}$$

where:

S = Total number of stories in the building, except $S = 13$ for buildings of 13 stories or less.

Response spectra were developed from the strong-motion records obtained from the Imperial Valley earthquake of 1940. These spectra were used as a basis for estimating response up to the time of the February 9 earthquake.

In 1952, a joint committee of the San Francisco section of the American Society of Civil Engineers (ASCE) and the Structural Engineers Association of Northern California published a paper recommending a code in which the coefficients (C) were related to the estimated or calculated fundamental period of the structure. This represented one of the first direct attempts to codify factors representing the dynamic behavior of structures in earthquakes. Portions of these recommendations are:

$$C = \frac{K}{T}$$

where:

C = Coefficient,
K = 0.015 for buildings,
K = 0.025 for structures other than buildings, and
T = Period in seconds.

For buildings $C_{max} = 0.06$, $C_{min} = 0.02$.
For other structures $C_{max} = 0.10$, $C_{min} = 0.03$.

These coefficients (C) were usually applied to dead loads plus 25 percent of live loads to arrive at a

lateral design force. In the case of storage or warehouse areas, 50 percent of live loads was added to dead loads and multiplied by (C).

In 1957, the Structural Engineers Association of California (SEAOC) realized the desirability of a uniform seismic code for California, and a committee of this association worked for 2 years to develop such a code.

This code was adopted by SEAOC in 1959 as *Recommended Lateral Force Requirements* and was published with a commentary in 1960. A reprint of these recommendations (SEAOC Code) follows. Some minor editing has been done to remove reference to sections of the commentary that are not included here.

RECOMMENDED LATERAL FORCE REQUIREMENTS
SEISMOLOGY COMMITTEE
STRUCTURAL ENGINEERS ASSOCIATION OF CALIFORNIA

December 1959

Sec. 2313. (a) **General.** These lateral force requirements are intended to provide minimum standards as design criteria toward making buildings and other structures earthquake-resistive. The provisions of this section apply to the structure as a unit and also to all parts thereof, including the structural frame or walls, floor and roof systems, and other structural features.

The provisions incorporated in this section are general and, in specific cases, may be interpreted as to detail by rulings of the building official in order that the intent shall be fulfilled.

Every building or structure and every portion thereof, except Type V buildings of Group I occupancy which are less than twenty-five feet (25′) in height, and minor accessory buildings, shall be designed and constructed to resist stresses produced by lateral forces as provided in this section. Stresses shall be calculated as the effect of a force applied horizontally at each floor or roof level above the foundation. The force shall be assumed to come from any horizontal direction.

(b) **Definitions.** The following Definitions apply only to the provisions of this section.

SPACE FRAME is a three dimensional structural system composed of interconnected members, other than shear or bearing walls, laterally supported so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

SPACE FRAME—VERTICAL LOAD-CARRYING: a space frame designed to carry all vertical loads.

SPACE FRAME—MOMENT RESISTING: a vertical load-carrying space frame in which the members and joints are capable of resisting design lateral forces by bending moments. This system may or may not be enclosed by or adjoined by more rigid elements which would tend to prevent the space frame from resisting lateral forces.

BOX SYSTEM is a structural system without a complete vertical load-carrying space frame. In this system the required lateral forces are resisted by shear walls as hereinafter defined.

SHEAR WALL is a wall designed to resist lateral forces parallel to the wall. Braced frames subjected primarily to axial stresses shall be considered as shear walls for the purpose of this definition.

(c) **Symbols and Notations.** The following symbols and notations apply only to the provisions of this section.

C	=	Numerical coefficient for base shear as defined in Section 2313 (d) 1.
C_p	=	Numerical coefficient as defined in Section 2313 (d) 2 and as set forth in Table No. 23-D.
D	=	The dimension of the building in feet in a direction parallel to the applied forces.
F_a	=	Allowable axial stress.
f_a	=	Computed axial stress.
F_b	=	Allowable bending stress.
f_b	=	Computed bending stress.
F_p	=	Lateral forces on the part of the structure and in the direction under consideration.
F_x	=	Lateral force applied to a level designated as x.
H	=	The height of the main portion of the building in feet above the base.
h_x	=	Height in feet above the base to the level designated as x.
J	=	Numerical coefficient for base moment as defined in Section 2313 (h).
K	=	Numerical coefficient as set forth in Table 23-C.
Σwh	=	Summation of the products of all $w_x \cdot h_x$ for the building.
M	=	Overturning moment at the base of the building or structure.
N	=	Total number of stories above exterior grade.
T	=	Fundamental period of vibration of the building or structure in seconds in the direction under consideration.
V	=	Total lateral load or shear at the base.
W	=	Total dead load. *EXCEPTION:* W shall be equal to the total dead load plus 25 per cent of the floor live load in storage and warehouse occupancies.
W_p	=	The weight of a part or portion of a structure.
w_x	=	That portion of W which is located at or is assigned to the level designated as x.

(d) **Minimum Earthquake Forces for Buildings. 1. Total lateral force and distribution of lateral force.** Every building shall be designed and constructed to withstand minimum total lateral seismic forces assumed to act non-concurrently in the direction of each of the main axes of the building in accordance with the following formula:

$$V = KCW$$

The value of K shall be not less than that exhibited in Table 23-C. The value of C shall be determined in accordance with the following formula:

$$C = \frac{0.05}{\sqrt[3]{T}}$$

EXCEPTION: C = 0.10 for all one and two story buildings.

T is the fundamental period of vibration of the structure in seconds in the direction considered. Properly substantiated technical data for establishing the period T for the contemplated structure may be submitted.

In the absence of such data, the value of T shall be determined by the following formula:

$$T = \frac{0.05\,H}{\sqrt{D}}$$

EXCEPTION: T = 0.10 N in all buildings in which the lateral resisting system consists of a moment-resisting space frame which resists 100% of the required lateral forces and which frame is not enclosed by or adjoined by more rigid elements which would tend to prevent the frame from resisting lateral forces.

For the purpose of computing C the value of T need not be less than 0.10 second.

The total lateral force "V" shall be distributed over the height of the building in accordance with the following formula:

$$F_x = \frac{V\,w_x\,h_x}{\Sigma wh}$$

EXCEPTION 1: One and two story buildings shall have uniform distribution.

EXCEPTION 2: Where the height to depth ratio of a lateral force-resisting system is equal to or greater than five to one, 10 per cent of the total force "V" shall be considered as concentrated at the top story. The remaining 90 per cent shall be distributed as provided for in the above formula.

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution on that level.

2. **Lateral force on parts or portions of buildings or other structures.** Parts or portions of buildings or structures and their anchorage shall be designed for lateral forces in accordance with the following formula:

$$F_p = C_p W_p$$

Table 23-C.—Horizontal force factor "K" for buildings or other structures [2]

Type or Arrangement of Resisting Elements	Value of K[1]
All building framing systems except as hereinafter classified.	1.00
Buildings with a box system as defined in Section 2313 (b).	1.33
Buildings with a complete horizontal bracing system capable of resisting all lateral forces, which system includes a moment resisting space frame which, when assumed to act independently, is capable of resisting a minimum of 25% of the total required lateral force.	0.80
Buildings with a moment resisting space frame which when assumed to act independently of any other more rigid elements is capable of resisting 100% of the total required lateral forces in the frame alone.	0.67
Structures other than buildings and other than those listed in Table 23-D.	1.50

(1) The coefficients determined here are for use in the State of California and in other areas of similar earthquake activity. For areas of different activity, the coefficient may be modified by the building official upon advice of seismologists and structural engineers specializing in aseismic design.

(2) Where wind load . . . would produce higher stresses, this load shall be used in lieu of the loads resulting from earthquake forces.

The values of C_p are in Table 23-D. The distribution of these forces shall be according to the gravity loads pertaining thereto.

3. **Pile foundations.** Individual pile footings of every building or structure shall be so interconnected by ties each of which can carry by tension and compression a horizontal force equal to 10% of the larger pile cap loading unless it can be demonstrated that equivalent restraint can be provided by other means.

(e) **Distribution of Horizontal Shear.** Total shear in any horizontal plane shall be distributed to the various resisting elements in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm as well as the rigidities of the vertical resisting elements.

(f) **Drift.** Lateral deflections or drift of a story relative to its adjacent stories shall be considered in accordance with accepted engineering practice.

(g) **Horizontal Torsional Moments.** Provisions shall be made for the increase in shear resulting from the horizontal torsion due to an eccentricity between the center of mass and the center of rigidity. Negative torsional shears shall be neglected. In addition, where the vertical resisting elements depend on diaphragm action for shear distribution at any level, the shear resisting elements shall

Table 23-D.—Horizontal force factor "C_p" for parts or portions of buildings or other structures

Part or Portion of Buildings	Direction of Force	Value of C_p
Exterior bearing and non-bearing walls, interior bearing walls and partitions, interior nonbearing walls and partitions over ten feet (10′) in height, masonry fences over six feet (6′) in height.	Normal to Flat Surface	0.20
Cantilever parapet and other cantilever walls, except retaining walls.	Normal to Flat Surface	1.00
Exterior and interior ornamentations and appendages.	Any Direction	1.00
When connected to or a part of a building: towers, tanks, towers and tanks plus contents, chimneys, smoke stacks, and penthouses. Elevated tanks plus contents not supported by a building.	Any Direction	0.20 [1]
When resting on the ground: tank plus effective mass of its contents.	Any Direction	0.10
Floors and roofs acting as diaphragms.[2]	Any Direction	

(1) When H/D of any building is equal to or greater than 5 to 1 increase value by 50%.

(2) Floors and roofs acting as diaphragms shall be designed for a minimum value of C_p of 10% applied to loads tributary from that story unless a greater value of C_p is required by the basic seismic formula $V = KCW$.

be capable of resisting a torsional moment assumed to be equivalent to the story shear acting with an eccentricity of not less than five percent of the maximum building dimension at that level.

(h) **Overturning.** Every building or structure shall be designed to resist the overturning effects caused by the wind forces and related requirements . . . or the earthquake forces specified in this section, whichever governs.

EXCEPTION: The axial loads from earthquake force on vertical elements and footings in every building or structure may be modified in accordance with the following provisions:

(1) The overturning moment (M) at the base of the building or structure shall be determined in accordance with the following formula:

$$M = J\Sigma F_x\, h_x$$

WHERE
$$J = \frac{0.5}{\sqrt[3]{T^2}}$$

The required value of J shall be not less than 0.33 nor more than 1.00.

(2) The overturning moment (M_x) at any level designated as x shall be determined in accordance with the following formula:

$$M_x = \frac{H - h_x}{H} M$$

At any level the overturning moments shall be distributed to the various resisting elements in the same proportion as the distribution of the shears in the resisting system. Where other vertical members are provided which are capable of partially resisting the overturning moments, a redistribution may be made to these members if framing members of sufficient strength and stiffness to transmit the required loads are provided.

Where a vertical resisting element is discontinuous, the overturning moment carried by the lowest story of that element shall be carried down as loads to the foundation.

(i) **Setbacks.** Buildings having setbacks wherein the plan dimension of the tower in each direction is at least 75 per cent of the corresponding plan dimension of the lower part may be considered as a uniform building without setbacks for the purpose of determining seismic forces.

For other conditions of setbacks the tower shall be designed as a separate building using the larger of the seismic coefficients at the base of the tower determined by considering the tower as either a separate building for its own height or as part of the overall structure. The resulting total shear from the tower shall be applied at the top of the lower part of the building which shall be otherwise considered separately for its own height.

(j) **Structural Frame.** Buildings more than 13 stories or one hundred and sixty feet (160′) in height shall have a complete moment resisting space frame capable of resisting not less than 25 per cent of the required seismic load for the structure as a whole. The frame shall be made of a ductile material or a ductile combination of materials. The necessary ductility shall be considered to be provided by a steel frame with moment resistant connections or by other systems proven by tests and studies to provide equivalent energy absorption.

(k) **Design Requirements. 1. Combined axial and bending stresses in columns forming a part of a space frame.** Maximum allowable extreme fiber stress in columns at intersection of columns with floor beams or girders for combined axial and bending stresses shall be the allowable bending stress for the material used. Within the center one-half of the unsupported length of the column, the combined axial and bending stresses shall be such that

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \text{ is equal to or less than 1.}$$

When stresses are due to a combination of vertical and lateral loads, the allowable unit stresses may be increased . . . .

2. **Building separations.** All portions of structures shall be designed and constructed to act as an integral unit in resisting horizontal forces unless separated structurally by a distance sufficient to avoid contact under deflection from seismic action or wind forces.

3. **Minor alterations.** Minor structural alterations may be made in existing buildings and other structures, but the resistance to lateral forces shall be not less than that before such alterations were made, unless the building as altered meets the requirements of this section of the Code.

4. **Unreinforced masonry.** All elements within the structure which are of masonry or concrete and which resist seismic forces or movement shall be reinforced so as to qualify as reinforced masonry or concrete. . . .

5. **Combined vertical and horizontal forces.** In computing the effect of seismic force in combination with vertical loads, gravity load stresses induced in members by dead load plus design live load, except roof live load, shall be considered.

The 1960 SEAOC recommendations are significant since they represented the thinking of the leading United States authorities in earthquake engineering at that time and recognized the fundamental differences in actual earthquake behavior of different framing systems by inclusion of the K factor. Also, these recommendations served as the basis for the Los Angeles City, Los Angeles County, and Uniform Building Codes, which were generally in use at the time most of the modern structures discussed in this report were designed.

Since 1960, many changes have been made in the SEAOC Code, most of which have been incorporated into other California codes. Following is a summary of the two significant changes to the 1960 SEAOC recommendations:

In 1966, ductile reinforced concrete was defined and accepted for buildings over 160 feet in height. A distinction was made at this time between space frame-moment resisting and space frame-ductile moment resisting. Also, the use of K values of 0.80 and 0.67 required a ductile moment-resisting space frame capable of resisting 25 percent or 100 percent respectively of the total lateral forces for all building heights.

In 1970, the overturning reduction factor, J factor, was eliminated from the SEAOC recommendations (Section 2313 (h), SEAOC Code).

In 1947 and 1965, two important additions were made to the Los Angeles city and some other southern California codes. These additions have resulted in increased life safety and valuable strong-motion data.

The first addition, in 1947, was retroactive and required that dangerous parapets and appendages adjacent to public ways be removed or adequately anchored. This work was virtually completed in Los Angeles before the February 9 earthquake and results are encouraging. This subject is treated at length in the paper "Unreinforced Masonry Buildings" in this volume.

An ordinance requiring strengthening or removal of dangerous parapets and appendages was adopted by the city of San Francisco but has not been implemented due to lack of funds.

The other addition, in 1965, required the installation of strong-motion instruments in major buildings. Three instruments were required in each building—one in the basement, one at midheight, and one at the roof. At the time of the San Fernando earthquake, 66 buildings were instrumented and usable records were obtained from all three instruments in about 25 buildings. The section on "High-Rise Buildings With Strong-Motion Instruments—Dynamic Analyses" in this volume includes dynamic analyses of several of these structures.

PHILOSOPHY OF CALIFORNIA EARTHQUAKE CODES

The philosophy, or intent, of California earthquake codes, as stated by the Structural Engineers Association of California (SEAOC) in a 1967 commentary reads as follows.

(a) **General.** The primary function of a building code is to provide minimum standards to assure public safety. Requirements contained in such codes are intended to safeguard against major failures and loss of life. Some owner-sponsored codes go farther than this. For example, Chapter 21, Title 24, of California's Administrative Code, related to the design and construction of public school buildings has as its added purpose the protection of property. This code is interested in minimizing damage as well as protecting the occupants. However, this is not the purpose of building codes generally.

The SEAOC Code is intended to provide criteria to fulfill the purposes of building codes generally. More specifically with regard to earthquakes, structures designed in conformance with the provisions and principles set forth therein should be able to:

1. Resist minor earthquakes without damage;
2. Resist moderate earthquakes without structural damage, but with some nonstructural damage;
3. Resist major earthquakes, of the intensity of severity

of the strongest experienced in California, without collapse, but with some structural as well as nonstructural damage.

In most structures it is expected that structural damage, even in a major earthquake, could be limited to repairable damage. This, however, depends upon a number of factors, including the type of construction selected for the structure.

It is to be understood that damage due to earth slides such as those that occurred in Anchorage, Alaska, or due to earth consolidation such as occurred in Niigata, Japan, would not be prevented by conformance with the SEAOC Code. The SEAOC Code has been prepared to provide minimum required resistance to typical earthquake ground shaking, without slides, subsidence, or faulting in the immediate vicinity of the structure.

After 10 years of minor changes since 1959, many engineers felt that the entire SEAOC Code should be reviewed and revised where necessary to keep pace with knowledge gained.

As a result, in 1970, the SEAOC board of directors established an ad hoc committee, consisting of the members of the 1970 Seismology Committee, together with the 1959 members who wrote the first code. The task of this ad hoc committee was to study in depth the basic design criteria of the SEAOC recommendations and commentary.

The ad hoc committee considered the behavior of earthquake-resistant construction in all past earthquakes, including San Fernando (1971).

Experience in recent earthquakes has led to the conclusion that the SEAOC Code no longer corresponds to the buildings being designed today and is not fulfilling its philosophy.

This basic conclusion, after considerable study, led the committee to several conclusions and recommendations which follow. These have been abridged by the authors.

Soil Conditions

Not specifically asked in the charge to the Committee but generally commented upon, is the necessity for considering the effect of the underlying soils. It was practically unanimously agreed that this should be considered in any future codes.

The form of the foundation effect must not be a blanket factor as used in the old Uniform Code (UBC), but must recognize the varying building response as influenced by underlying soils and earthquake type, and should be included in the best form possible based on available knowledge.

Elastic and Inelastic Analysis

The Committee strongly recommends that structures be designed under the philosophy of two levels of performance, elastic for service levels and the frequent earthquakes that cause the building to respond at low levels; and inelastic for the ultimate energy demands resulting from major earthquakes and the inelastic work capacity to prevent collapse. Inelastic methods should account for all known conditions such as degrading hysteresis curves, $P-\Delta$ effects and the loss of strength of material under cyclic loadings.

The Committee urges that elastic design principles be used for service loads.

Materials of Construction

The performance of the structure is not alone determined in the "Loads" Section of the Code. It is based also on the material sections.

The Committee and the general membership were strongly in favor of load factor design principles. However, there was very strong agreement that present load factors in the SEAOC Code are not correct. In general, they are too low.

If load factor design is to be used, load factors should be revised to reflect: (1) type of material; (2) workmanship; (3) amount of research available; (4) history and experience; (5) variability in place; (6) performance in cyclic loading and, in general, load factors must be larger or the predicted ultimate capacities must be lower.

Relative design stresses have greatly increased. Effective load factors in many cases have greatly decreased from the working stress design methods used in 1959 to the new ACI [American Concrete Institute] 318–71 Code. Present design practices in some instances have doubled the allowable loads on concrete columns in biaxial bending. This has had the practical effect of halving of lateral force requirements on concrete buildings in less than 10 years—at a time when experience dictates that we increase our standards of performance. Material stresses and load factors and details must be tied to the Code force levels.

Damage Control

The Committee recommends that there should be no reduction in the damage control provisions of the Code. Most Committee members say that there either should be more control or we should keep the present balance. At any rate, the Code and the Commentary must be revised to state what the engineer can or cannot deliver, regardless of code. It is obvious that we cannot assure against earthquake damage in even moderate or smaller earthquakes. A statement about possible damage should be in the Code—not only in the Commentary—because that is the legal document upon which liability may be based. The Commentary itself must be more explicit and more detailed about the probability of damage in the unanimous opinion of the Committee. Controlling drift under major intensity earthquakes may be one answer to satisfying the operative criteria for emergency structures, and can lead toward structural and nonstructural damage control under elastic conditions.

Design Forces

The Committee discussed the level of design forces presently set forth in the Code and later voted on the

question as to whether force levels should be increased or decreased. Not one vote was cast for *decreased* forces. The majority voted for increased forces. A large minority voted to keep them the same. Many restated the question and agreed that the safety and performance must be increased but realized that increased forces per se did not necessarily increase performance. It is recommended that a judicious increase of forces in some types of buildings be required and an increase in ductility be required in others. An increase in performance and safety is definitely the objective. If ductility is not assured, then major force increases are necessary.

Use Factor

The function of a building or structure must be recognized in the level of performance specified by the Code, using public safety criteria. It is most important that we insist that those structures housing emergency facilities that would be required for use after a major earthquake be so designed and constructed that they remain functional after the disaster. These structures would include hospitals, centers of emergency, communications and control, and evacuation facilities.

Reevaluate K *Factors*

The Committee recommends that the present K values be retained in the Code, but must be reevaluated. Since ductility and stability are of prime importance in the earthquake performance of a structure, some way should be found to measure and specify ductility as required to perform and as used in the system. The K factor should reflect: (1) experience with different types of framing; (2) the ductility of the system; (3) advantages of a secondary system or reserve strength; (4) basic stability; (5) redundancy; and (6) basic performance of materials. It should not be confused with C, Z, or load factors. It is not a measure of factor of safety, hedge against poor workmanship, nor a measure of amount of research.

The Code must reflect the changing types of buildings and framing used and the following items must be considered when setting design requirements and force levels:

The average building today has virtually eliminated redundancy. Concrete exterior walls have disappeared in high-rise construction in favor of curtain walls. Interior permanent walls are now lightweight construction and partitions are kept floating. There is little or no "uncalculated" strength. The Committee is most concerned that the very necessary attribute of redundancy in earthquake-prone areas has often been disregarded. Redundancy must be either required or a much higher level of forces must be required to reflect the actual experience record of force levels which have occurred in recent earthquakes as shown by some instrumentation. In the $K = 0.67$ (frame) buildings of 1960 it was expected that essentially all of the columns and girders would be moment resisting. If one element failed, others could pick up the load. Today, many high-rise buildings have a small proportion of moment-resisting connections. The validity of designs using $K = 0.67$, as frequently used, is open to serious question. There are very inconsistent earthquake design criteria now required for buildings of different types (types as defined by K values).

In 1960 most steel buildings were riveted or bolted. Now most tall structures are welded. In most respects, the welding is vastly superior, but some details, heavy column splices, for example, have lost all semblance of ductility. Damping characteristics have been reduced, increasing the spectral response potential.

Neglected Factors in Present Code

Our present Code has been essentially silent in certain areas of concern, which greatly affects the performance of the building and the safety of the occupants. It is recommended that adequate provisions be included in the next code, such as the following:

1. Some criteria are needed to provide extra strength at discontinuities of stiffness or mass. The performance of buildings on "stilts" has been inadequate, to say the least. Possibly a change in story stiffness over a certain amount should require a certain specified increase in strength capability or ductility.

2. The Code should be more explicit about the need for "tying together" collector members, chords, reinforcement around openings, the necessity of a complete continuous stress path, etc. One item frequently overlooked is the development of a reasonable stress distribution across the width of diaphragm.

3. All recent earthquakes have emphasized the importance of adequate bracing and anchorage of architectural features, curtain walls, lights, ceilings, mechanical and electrical equipment, and better design practices for elevators. Possibly the structural engineers should set criteria for the architects, electrical engineers, mechanical engineers, and equipment suppliers to use.

4. The Code must recognize that vertical accelerations due to either horizontal or vertical ground motion can have a major bearing on the performance of some elements of a structure.

5. Possibly most important of all, in the light of recent and not-so-recent failures, the Code must recognize the requirements of deformation control not only in the elastic state, but especially in the plastic, ductile, damaged and deformed state, which is the true state of affairs after a major earthquake. The control of deformations after failure has been reached will determine whether the structure merely failed or whether it has "pancaked" with the consequent loss of life.

The following comments by the authors are intended to amplify and add neglected factors to the above.

In order for some buildings designed in accordance with the SEAOC Code to survive a major earthquake, some portions of the structure will be stressed beyond the elastic limit. This means that deformations will continue with little or no increase in the force levels. The duration of the earthquake then becomes a significant factor.

For a short-duration earthquake, a structure might survive without collapse even when subjected to high force levels. On the other hand, a prolonged strong shaking may result in the collapse of the same structure.

To properly evaluate the amount of deformation a structure will undergo, it is necessary that not only the force level, but also the duration of strong motion, be considered. When the force level results in stresses within the plastic range, the actual magnitude of the forces is limited by the capacity of the system.

The assumption, based on present criteria, that portions of the structure must go beyond the elastic range, emphasizes the importance of providing ductility in these elements. A sudden or "brittle" type failure is not acceptable if it results in a collapse of any portion of the structure. Actual deflections in the plastic range during a large earthquake must be examined. Although methods are available for calculating these deflections, further development of better analytical techniques should be pursued.

The calculation of actual deflections is needed to realistically determine structural separations between buildings and between nonstructural elements and frames as well as $P-\Delta$ and hysteresis effects.

One very important factor is the necessity for good construction practices in order to produce earthquake-resistant structures. This requires that the structural engineer observe the construction and review shop drawings to assure that the construction documents are being correctly interpreted and followed. This need is especially critical for the complex details of modern ductile frame high-rise construction. This degree of observation results in much more work on the part of the structural engineer than has been provided in the past.

Another important factor, not included in most codes, is the consideration of the hazard of old non-earthquake-resistive structures. Old masonry-bearing wall structures constructed without reinforcing and with poor quality mortar have resulted in the loss of lives in both moderate and severe earthquakes. The problem of the removal or strengthening of these old buildings is mainly economic. There are probably 20,000 such structures in Los Angeles city alone, and perhaps 100,000 in UBC, Zone 3 within the United States.

In early 1971, the Earthquake Engineering Research Institute (EERI), American Society of Civil Engineers (Los Angeles Section), and the American Institute of Architects (Los Angeles Chapter) adopted resolutions recommending that the State of California adopt legislation to require the systematic correction or removal of old buildings. The SEAOC had already adopted a similar resolution at their annual convention in October 1970. A press conference to release the EERI and SEAOC resolutions was scheduled for February 9, 1971. Needless to say, this conference was not held.

OTHER ORGANIZATIONS ACTIVE IN THE CODE FIELD

Several industry organizations are active in the code field. The American Concrete Institute (ACI) publishes the principal code concerning concrete known as ACI 318. Portions of this code are incorporated in the concrete sections of California and other codes.

The American Institute of Steel Construction (AISC) publishes the dominant code for steel as part of their design handbook. This code is also incorporated in the steel sections of most codes.

Other similar organizations are active in the concrete, masonry, steel, wood, and light-gauge metal fields.

The American Society for Testing Materials (ASTM) dominates the testing and certification field in the United States for all materials. They develop testing standards for most construction materials. References to testing procedures are usually given as ASTM, followed by letters and numbers which identify the particular procedure and materials (i.e., ASTM A-36).

ELEMENTS OF LATERAL FORCE DESIGN

Definitions

This section is intended to explain the meanings of some of the terms used by California civil and structural engineers in the analysis of earthquake-resistant structures. It is not presumed that this be a design manual nor that it cover all of the judgment factors involved.

The following are definitions of pertinent terms used in subsequent discussions and reports. These definitions are quoted from *Recommended Lateral Force Requirements and Commentary,* Seismology

Committee, SEAOC, 1967. Minor editing was done by the authors for clarity.

Definitions. In the evolution of design criteria for earthquake-resistant design, the type of a structural system has been introduced directly as a consideration in stipulating lateral earthquake design forces. The concept is not new; structural engineers have long recognized that some types of construction are inherently more earthquake resistant than others. It has required, however, the definition of some new terms:

SPACE FRAME (quoting from the SEAOC Code) "is a three dimensional structural system composed of interconnected members, other than bearing walls, laterally supported so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems." This definition is intended to be general enough to permit members to be sloped or battered as well as horizontal and vertical, so as not to exclude special space structures. Usually, space frames are composed of horizontal beams or girders and vertical columns. There may or may not be diagonal members associated with the space frame, such as knee-braces, rod-bracing, X-bracing, etc.

SPACE FRAME—VERTICAL LOAD-CARRYING is "a space frame designed to carry all vertical loads." The frame may or may not be moment-resisting. The words "complete" (as related to space frame) and "all vertical loads" (as related to space frame—vertical load-carrying) are not to be construed in an absolute sense. Accordingly, where these words appear in this Commentary, they will be modified, or be understood to be modified, with the word "substantially." The reasoning here is that the action of a multi-storied building is not significantly influenced by the presence of a minor portion of bearing walls—around a stairwell, for example. Also, in a tall building with setbacks, the completeness of the frame for the tower, when carried through to the foundation, is not adversely affected by bearing walls in the base structure adjacent to the tower. Neither does it seem reasonable to require that basement walls be frame-supported; nor walls of not more than one story that are supported directly on foundation walls.

SPACE FRAME—MOMENT RESISTING is a vertical load-carrying space frame in which the members and joints, of that part of the space frame selected to be "moment-resisting," are capable of resisting design lateral forces by bending moments. This frame has members and joints designed to resist the bending moments corresponding to a set of stipulated or assumed proportion of the prescribed lateral forces. This system may or may not be enclosed by or adjoined by more rigid elements which would tend to prevent the space frame from resisting lateral forces. The design and construction of the frame to resist bending moments may or may not have any relation to its ability to receive the design load because of more rigid elements which are in the structure or which may encase the frame.

However, in the case of both "Moment Resisting Space Frames" and "Ductile Moment Resisting Space Frames," defined below, it is essential that it be shown that neither the elastic nor inelastic action including failure of the more rigid elements, will impair the vertical or lateral load resisting ability of the space frame.

SPACE FRAME—DUCTILE MOMENT RESISTING is a moment-resisting space frame of structural steel (ASTM A-7, A-36 or A-441) or of special reinforced concrete. . . . The use of these frames qualifies the building for $K = 0.67$ or $K = 0.80$ earthquake design factors for *all* heights, without limit. Shear walls used in conjunction with ductile moment-resisting space frames in a $K = 0.80$ building must conform to . . . the SEAOC Code or be composed of axially loaded bracing members of ASTM A-7, A-36 or A-441 structural steel.

BOX SYSTEM is a structural system without a substantially complete vertical load-carrying space frame. In this system, the required lateral forces are resisted principally by shear walls as hereinafter defined. It is a composite system of vertical load-carrying framing, bearing walls, and perhaps other lateral stiffening shear walls. The structure may have some columns, but generally columns in conjunction with bearing walls. Shear walls may also be bearing walls. Horizontal elements which distribute the lateral forces between the masses accelerated by the earthquake and the vertical resisting elements (shear walls) may be diaphragms of any of several materials, or horizontal bracing trusses. In summary, a box system is characterized by all of the following: 1. incomplete vertical load-carrying space frame, 2. bearing walls carrying part or all of the vertical loads, 3. lateral forces resisted by shear walls, and 4. horizontal distributing system consisting of diaphragms or bracing trusses [fig. 1].

SHEAR WALL "is a wall designed to resist lateral forces parallel to the wall. Braced frames subjected primarily to axial stresses shall be considered as shear walls for the purpose of this definition." A shear wall is normally vertical, although not necessarily so.

LATERAL FORCE-RESISTING SYSTEM is "that part of the structural system to which the lateral forces prescribed in Section 2313 (d) (SEAOC Code) are assigned" by the structural engineer. The entire space frame need not be part of the lateral force-resisting system, but the latter must be completely stable in all directions, independent of other space frame elements or shear walls that may be attached thereto. Generally, this will mean not less than two frames in each direction corresponding to the two principal axes of the building or structure, and spaced far enough apart to assure stability.

DIAPHRAGM is essentially a horizontal girder composed of a web (such as a floor or roof slab) with adequate flanges, which distributes lateral forces to the vertical resisting elements. For the purposes of this Code, horizontal bracing trusses or systems must conform to the provisions applicable to diaphragms. A diaphragm may be inclined or curved, like a sloping or curved roof.

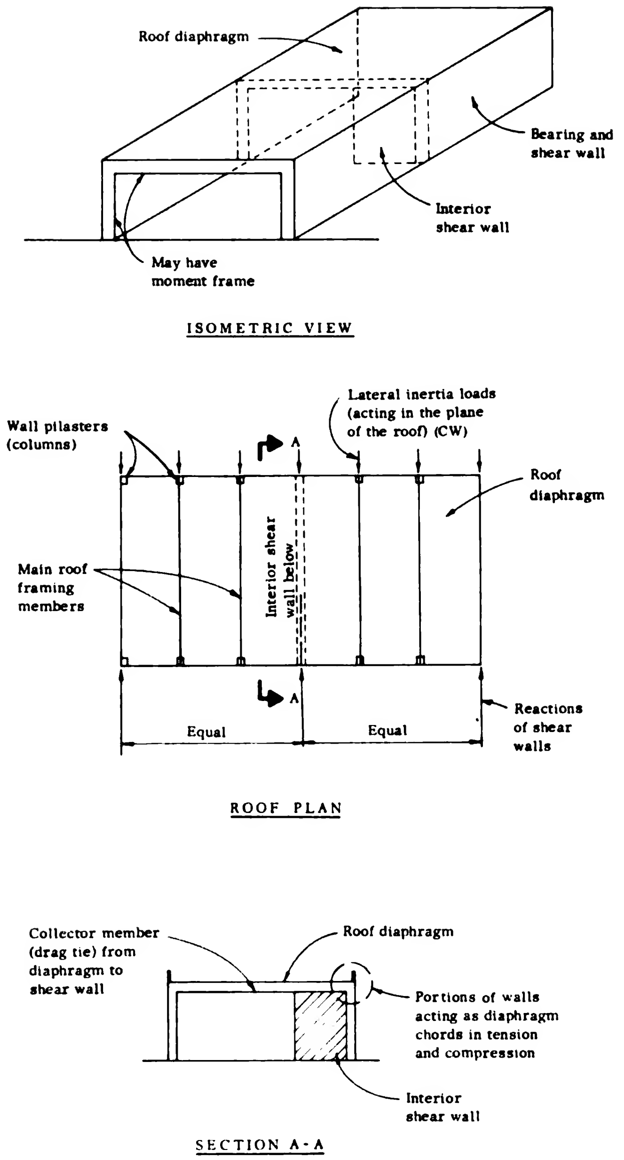

Figure 1.—Example of a box system.

DYNAMIC APPROACH is a simplified analysis which provides a rational basis for establishing equivalent static forces to simulate the conditions and stresses that will occur under complex earthquake ground motion. Rigorous dynamic analyses can be made for the effect of recorded ground motions on simplified structures and these are to be encouraged toward further improvements in practical design criteria for aseismic design. But the design criteria to be used in the day to day design of structures must not be so complex as to be impractical, nor so involved as to require a disproportionate part of the total design effort. At this stage of knowledge, the best that can be accomplished reasonably is to have the design criteria fairly consistent with the dynamic nature of the problem; hence, the term "dynamic approach."

STATIC FORCE EQUIVALENTS are a set of design static forces established to simulate the effects, in shears, moments, and direct stresses, of the erratic earthquake ground motion. It is to be noted that during an earthquake there are, in fact, no externally applied forces on a structure other than the base shear, base moment, and a base vertical force. The last is not specifically covered in the SEAOC Code, but is provided for by the requirement for the combination of stresses resulting from the full vertical design loads with those resulting from the prescribed seismic forces. Also special provisions are set forth where reductions in vertical load caused by vertical ground motion are important. In the SEAOC Code the design base shear is defined, and is resolved into static force equivalents.

BASE SHEAR is the total lateral earthquake design force on the structure in a particular direction being considered, which is generally normal to a principal axis (in plan) of the structure. The base shear is the horizontal force transmitted from the ground into the structure. The base shear, or the shear at any level, is the summation of the individual lateral forces from the top down to the base or to the level in question.

TRIANGULAR DISTRIBUTION is a method for resolving the base shear into static force equivalents applied laterally to the structure. Fundamentally, as the structure vibrates each mass is subjected to inertia forces. By Newton's Law, these inertia forces are proportional to mass times acceleration. When deflection is proportional to force as in the elastic range of action, for which the design criteria are established, the acceleration is proportional to the deflection of the mass. Hence, the inertia forces are proportional to mass times acceleration, and also to mass times deflection. Since the masses and their distribution are known, it is only necessary to know the shape of the deflection curve in order to have a means to distribute the base shear. It has been demonstrated that for an idealized uniform building vibrating in the fundamental mode, the shape of the deflection curve is essentially a straight line, zero at the bottom and maximum deflection at the top of the structure. If the mass is uniformly distributed over the height, the multiple of the equal masses times the linear deflection results in a triangular distribution of the base shear, zero at the bottom and maximum at the top. . . .

RESPONSE is the effect produced on a structure by earthquake ground motion. The spectral response is the maximum response during an earthquake. When a recorded ground motion is applied to a series of simple spring-mass structures, varying only by the natural period, the plots of the spectral responses constitute the earthquake spectra. These earthquake spectra may be determined without damping or with damping, usually of the viscous type. The spectra may be expressed as velocity spectra, acceleration spectra, displacement spectra, or other variables related to these units. In any event, they all express a response characteristic of the particular earthquake. Their development has been an outstanding accom-

plishment in engineering seismology and very useful in the application of a dynamic approach to code earthquake-resistant design criteria.

DAMPING is a rate at which a natural vibration decays. If a simple spring mass system were set in motion and had no damping, it would continue to vibrate ad infinitum. To some degree energy is lost and this energy loss results in a decreasing amplitude of vibration. In a forced vibration, such as that which might be induced by an earthquake, the effect of damping is to decrease significantly the magnitude of the response of the structure to the ground motion. For mathematical purposes, considering response in the elastic range only, it is usual to assume so-called "viscous damping" or damping proportional to velocity. In actual structures the nature of the damping is not so simple, as inelastic action takes place, especially in destructive earthquakes. Suffice it to say here that it is the combination of damping in the elastic range, inelastic action, and other factors that accounts for the good behavior of structures designed for modest lateral forces in rather severe earthquakes.

MODES: Simplified spring-mass systems have only one mode in which they can vibrate. Most real structures are capable of vibrating in several configurations, or modes, each with its own natural period. The elastic response of a structure capable of vibrating in several modes is the sum of the concurrent responses of each of these modes. It has been shown that each mode can be represented by a spring-mass system of period equal to that of the mode represented, and of a certain proportion of the total mass of the actual structure. Hence, the dynamic approach used in justifying the period criterion for base shear encompasses the analysis of the response of the modal spring-mass systems in somewhat idealized configurations.

TORSION: Structures vibrate in complex ways, involving translational vibrations and also torsional vibrations. Torsional vibrations, like translational vibrations, can occur in multiple modes. Torsional effects are most severe in unsymmetrical structures, but even symmetrical structures are subject to torsional vibration, and the SEAOC Code stipulates that provision be made for "accidental torsion" as well as torsion due to calculated eccentricities [fig. 2].

DRIFT (as used in lateral force design for wind or earthquake) has two connotations:

1. The lateral deflections, due to design forces of wind or earthquake, of any point in the structure relative to the ground, or the absolute deflection;

2. The incremental lateral deflection in any story due to the design forces of wind or earthquake. This concept is more properly the story drift, or the relative motion of the upper floor to the lower floor of any story.

OVERTURNING MOMENT is the moment on the structure as a whole at any given level, due either to wind or to earthquake lateral forces. The SEAOC Code restricts itself to criteria for determining the overturning moment due to earthquake.

Box System

Since most of the one-story buildings covered in this report are of the box-type framing, some additional comments on this construction are appropriate.

Figure 1 shows some typical details for box-type systems. Main roof framing members such as trusses, glued laminated wood beams, or tapered steel girders generally span across the narrow dimension of the building. Side (longitudinal) walls may have pilasters or columns built into the walls. These pilasters provide supports to the main roof framing members. In some cases, roof girders are supported at the walls by "corbels," which are short cantilever members projecting from the walls or columns. Roof support may also be provided by interior columns, depending on the framing system used and the building width.

The roof diaphragms may be of wood sheathing boards, plywood sheathing, metal decking, poured gypsum, concrete, horizontal bracing systems, or combinations of these. Most of the one-story buildings covered in this report had plywood roof diaphragms.

Walls may be reinforced concrete, often precast flat and lifted (tilted) up in place, reinforced brick, reinforced hollow concrete block, wood, or metal.

The roof diaphragm acts as a girder lying on its side with its web composed of the roof diaphragm material (plywood, metal deck, etc.) and its flanges consisting of the upper portions of the walls or special members at the edges of the diaphragm. These flanges (chords) resist tensile and compressive loads as the "girder" spans between "shear walls." The diaphragm resists the lateral loads and distributes them to the vertical resisting elements.

Interior shear wall elements may be used for excessively long buildings. These shear walls may extend only part way across the building. Under transverse lateral loading, this interior shear wall may resist a large portion of the loads and therefore must be well connected to the roof diaphragm. This generally requires the use of "collector" members which are connected to the full width of the diaphragm and well connected to the shear wall.

Many other configurations involve the use of "collector" members or "drag struts" as they are sometimes called. Design of these members and their connections is often neglected, and this can result in

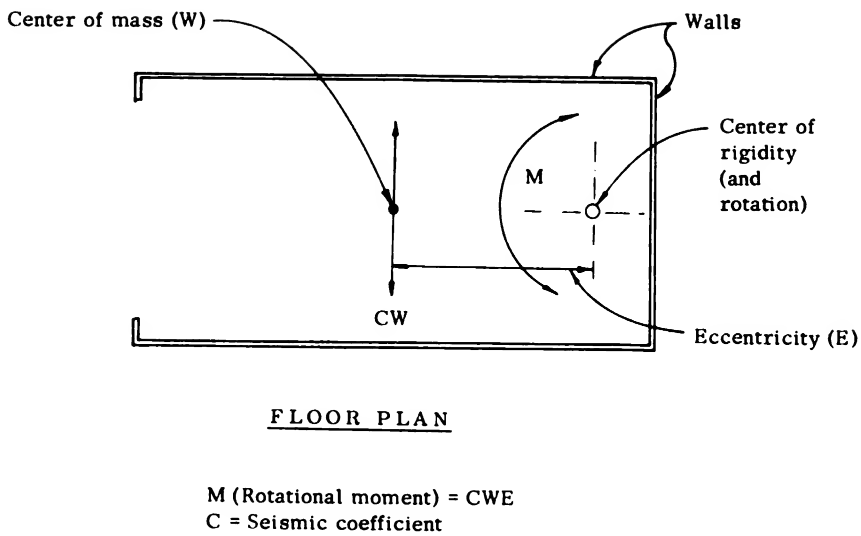

Figure 2.—Building rotation.

severe local damage in an earthquake and possible building collapse.

Building Rotation

Building rotation or torsion is often considered in the analysis of buildings to resist lateral forces. Rotation is the result of the eccentricity (distance) between the center of mass and the center of rigidity. The lateral force is applied at the center of mass and the structure is assumed to rotate about the center of rigidity (fig. 2). Rotation may be most severe in unsymmetrical structures.

The amount of rotation is also dependent on the rigidity of the diaphragm. For instance, relatively flexible diaphragms, such as wood sheathing and plywood, are not as capable of distributing rotational forces as rigid materials such as reinforced concrete. Therefore, rotation is often ignored in buildings with flexible diaphragms, and diaphragms often are assumed to have simple spans between shear walls. For example, referring to figure 1 and assuming a flexible diaphragm, the lateral roof load resisted by the interior shear wall is assumed to be 50 percent of the total roof load. The end walls are assumed to resist 25 percent each.

The large portion of the load which is transmitted to the interior shear wall emphasizes the need for collector members which extend completely across the building and are well connected to the roof diaphragms. The connections of the collector members to the shear wall are also critical. Achieving the desired ductility in these connections may be difficult or impractical. Perhaps the mode of failure should not involve these connections.

Symmetrical structures with simple geometrical plans generally perform better in earthquakes than those with irregular plans and unsymmetrical resisting elements. This does not mean that it is impossible to design the latter to perform equally well. However, the chances for mistakes in both design and construction are more likely in complicated, irregular structures. Also, more assumptions as to actual seismic behavior must be made for unsymmetrical designs.

SUMMARY

History has shown that California earthquake-resistive code provisions constantly are being reviewed, modified, and improved as experience is gained from each earthquake and from research. There are constant economic and political pressures being exerted which have influenced the codes. The subject is complex and requires that judgments be made by design engineers with extensive backgrounds in earthquake engineering. It is impossible, and even inadvisable, to codify all of the factors which must be considered.

The ultimate test of any earthquake-resistant design is provided by the earthquake itself. Each earthquake has been somewhat unique and has provided new lessons and data that must be examined and evaluated for their effect on code provisions.

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7. Coast and Geodetic Survey, *The Prince William Sound, Alaska, Earthquake of 1964 and Aftershocks,* Vol. II, Part A, Environmental Science Services Administration, U.S. Department of Commerce, Washington, D.C., 1967, pp. 196–215.

8. Coast and Geodetic Survey, *The Santa Rosa, California, Earthquakes of October 1, 1969,* Environmental Science Services Administration, U.S. Department of Commerce, Washington, D.C., 1970, pp. 57–59.

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Building Reports

SUBCOMMITTEE
ON
BUILDINGS

NOAA/EERI Earthquake Investigation Committee

Building reports were prepared for individual buildings and for groups of buildings at single locations. They are arranged by type of building construction or use in the following manner.

Earthquake-resistant buildings:
- Low-rise industrial and commercial buildings . . . . (1–15)
- Detention facilities . . . . . . (16–17)
- Hospitals and medical facilities . (18–26)
- High-rise buildings with strong-motion instruments—dynamic analyses . . . . . . (27–37)
- High-rise buildings—not instrumented . . . . (38)

Nonearthquake-resistant buildings:
- Unreinforced masonry buildings (39–43)

Veterans Administration Hospital . (44)

Public school buildings

Building report numbers were assigned to 44 buildings (indicated by numbers in parentheses). Table 1 lists the 44 buildings by their assigned numbers and gives the name of each, type of construction or use, and address. The accompanying map (fig. 1) shows their locations. Within the low-rise industrial and commercial grouping, the introductory reports for the Sylmar industrial tract and San Fernando industrial tract include detailed building location maps for those tracts. Location maps also are included in the paper on "Public School Buildings." Introductory reports provide "section contents" listing the papers that follow.

Table 1.—Building report numbers 1 through 44

Name [1]	Location (see fig. 1)
1. Stone's Liquor Store (LR)	15151 Bledsoe Street, Sylmar.
2. Warehouse Building (LR)	12884 Bradley Avenue, Sylmar.
3. All Phase Color (LR)	12874 Bradley Avenue, Sylmar.
4. Bell Metrics (LR)	12836 West Arroyo Avenue, Sylmar.
5. M&L Machine Shop (LR)	12424 Gladstone Avenue, Sylmar.
6. Vector Electronics (LR)	12460 Gladstone Avenue, Sylmar.
7. Wendell Machine Shop (LR)	12685-12691 Foothill Boulevard, Sylmar.
8. Bennett Industries (LR)	1647 Truman Street, San Fernando.
9. Thriftimart Market (LR)	24200 Lyons Avenue, Valencia, Los Angeles County.
10. W. T. Grants (LR)	19419 Soledad Canyon Road, Saugus, Los Angeles County.
11. Builder's Emporium (LR)	19407 Soledad Canyon Road, Saugus, Los Angeles County.
12. Alpha Beta Market (LR)	13570 Eldridge Avenue, Sylmar.
13. Boys Market (LR)	2040 Glenoaks Boulevard, San Fernando.
14. Museum for Antique Cars (LR)	15180 Bledsoe Street, Sylmar.
15. Goodwill Industries (LR)	1132 Pico Street, San Fernando.
16. San Fernando Valley Juvenile Hall (DF)	15900 Filbert Street, Sylmar.
17. Camp Karl Holton Juvenile Facilities (DF)	16691 North Little Tujunga Canyon Road, Los Angeles County.
18. Foothill Medical Center (HM)	12502 Van Nuys Boulevard, San Fernando.
19. Pacoima Lutheran Medical Center (HM)	11600 Eldridge Avenue, Pacoima.
20. Golden State Community Mental Health Center (HM)	Do.
21. Pacoima Memorial Lutheran Hospital (HM)	Do.
22. Indian Hills Medical Center (HM)	14935 Rinaldi Street, Los Angeles.
23. Holy Cross Hospital (HM)	15031 Rinaldi Street, Los Angeles.
24. Olive View Hospital, Medical Care Facility (HM)	14445 Olive View Drive, Sylmar.
25. Olive View Hospital, Psychiatric Unit (HM)	Do.
26. Olive View Hospital, Heating and Refrigeration Plant (HM)	Do.
27. Sheraton-Universal Hotel (HI)	3838 Lankershim Boulevard, Los Angeles.
28. Bank of California (HI)	15250 Ventura Boulevard, Los Angeles.
29. Holiday Inn (HI)	8244 Orion Avenue, Van Nuys.
30. Holiday Inn (HI)	1640 Marengo Street, Los Angeles.
31. Bunker Hill Tower (HI)	800 West First Street, Los Angeles.
32. KB Valley Center (HI)	15910 Ventura Boulevard, Los Angeles.
33. Muir Medical Center (HI)	7080 Hollywood Boulevard, Los Angeles.
34. Kajima International Building (HI)	250 East First Street, Los Angeles.
35. Certified Life Building (HI)	14724 Ventura Boulevard, Los Angeles.
36. Union Bank Square (HI)	445 South Figueroa Street, Los Angeles.
37. 1901 Avenue of the Stars Building (HI)	Century City, Los Angeles.
38. Union Bank (HN)	15233 Ventura Boulevard, Sherman Oaks.
39. Kress Store (NE)	1004 San Fernando Road, San Fernando.
40. Three-Story Loft Building (NE)	219 South Central Avenue, Los Angeles.
41. Mother Cabrini Girls School (NE)	601 North Hill Place, Los Angeles.
42. Midnight Mission (NE)	396 South Los Angeles Street, Los Angeles.
43. Cotton Exchange Building (NE)	106 West Third Street, Los Angeles.
44. Veterans Administration Hospital (HM)	13000 Sayre Street, Los Angeles County.

NOTE.—The communities of Sylmar, Pacoima, Van Nuys, Sherman Oaks, and Century City are all in the city of Los Angeles. The city of San Fernando is incorporated as a separate community. Locations noted in Los Angeles County are unincorporated.

[1] Abbreviations are defined as follows:

LR —Low-rise industrial and commercial building.
DF —Detention facility.
HM—Hospital or medical facility.
HI —High-rise building with strong-motion instrument.
HN—High-rise building, not instrumented.
NE —Nonearthquake-resistant building.

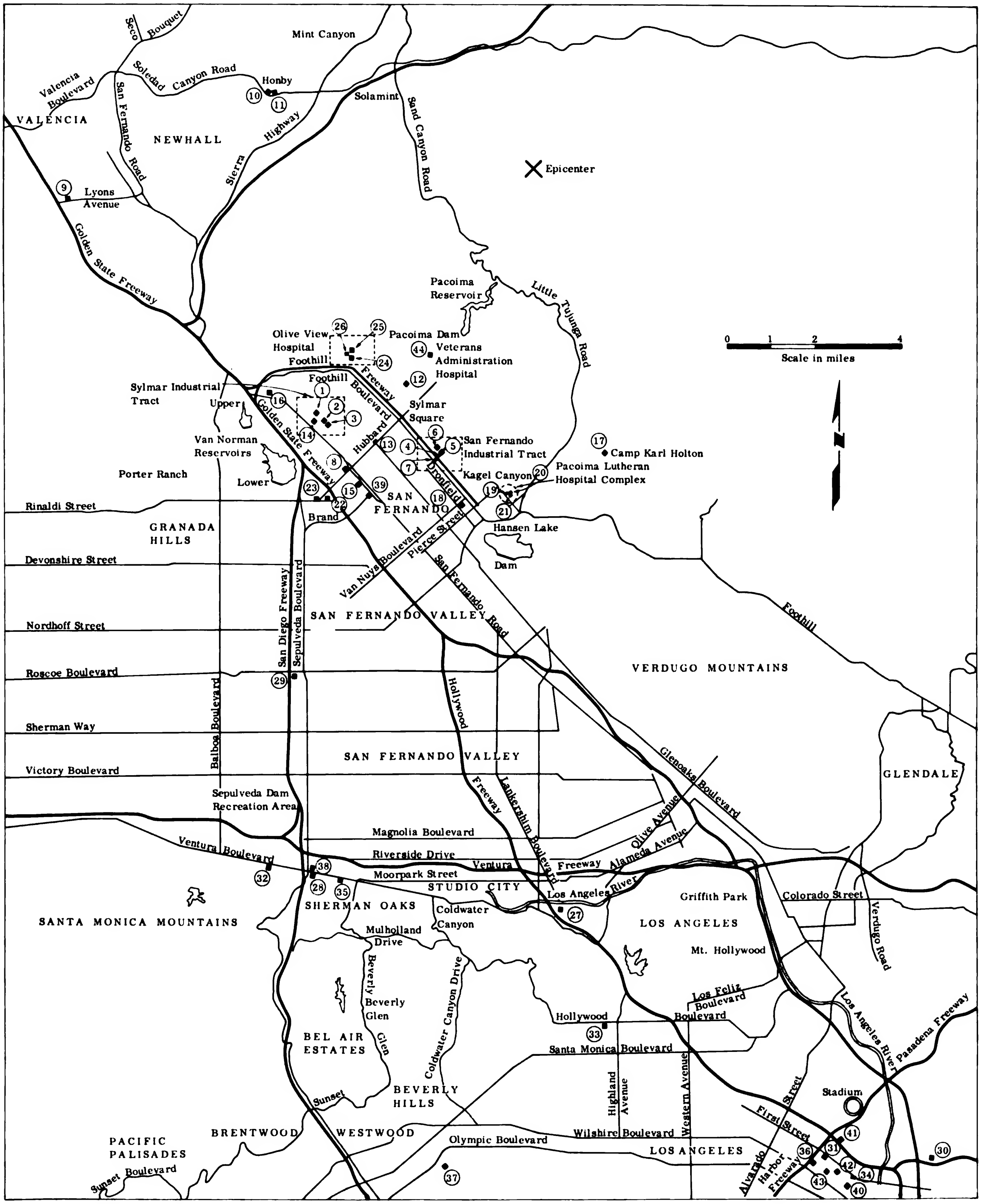

Figure 1.—Locations of buildings described in building reports 1 through 44.

Low-Rise Industrial and Commercial Buildings

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SUBCOMMITTEE ON BUILDINGS

NOAA/EERI Earthquake Investigation Committee

Much of the damage to buildings during this earthquake occurred to the so-called low-rise industrial and commercial buildings. Fifteen buildings of this type have been studied and are included in the section that follows. Reports on 13 one-story buildings with wood roof systems are presented first. Included in this group are three buildings in the Sylmar industrial tract, four buildings in the San Fernando industrial tract, and six buildings at other locations in the heavily shaken area. A description of local site conditions for each industrial tract is included in a short introduction to the group of buildings in the tract. The reports on the wood roof and masonry wall buildings are followed by a report on the Museum for Antique Cars, a five-story monolithic concrete building, and a report on Goodwill Industries, a one-story building with precast concrete roof and walls. All 15 buildings, located between 8 and 17 miles from the epicenter, were built since 1958, and all were required to be analyzed for earthquake forces as dictated by the applicable building codes. The damage observed from the February 9 earthquake and subsequent aftershocks varied from moderate to complete collapse of the structure. Although the damage was spectacular in many instances, no deaths or serious injuries were reported in these buildings because there were few, if any, occupants at the time of the earthquake due to the time of occurrence. Had the shock occurred even 1 hour later, the loss of life and injuries would have been significant.

The 13 buildings with the wood roof systems generally have plywood roof sheathing, wood or steel roof girders, and reinforced precast concrete or unit masonry walls. The roof construction used here often is called a panelized system. This system involves the use of prefabricated roof units consisting of 4- by 8-foot pieces of plywood nailed to 2- by 4-inch (nominal) wood members. The 2- by 4-inch

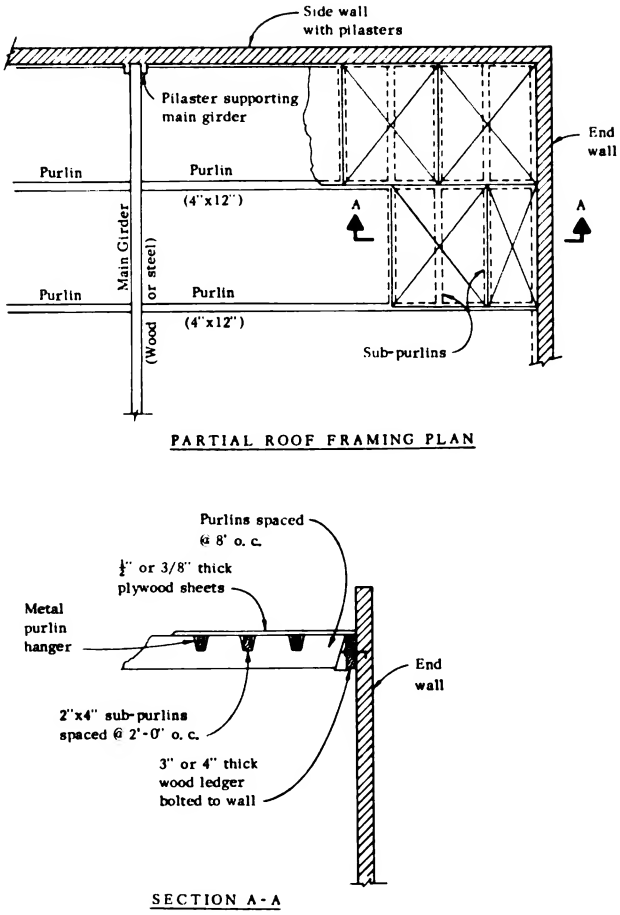

Figure 1.—Typical panelized wood roof details.

members serve as subpurlins, and are supported in sheet metal hangers at each end from larger (commonly 4- by 12-in.) purlin members that span between main roof girders (fig. 1). The 4- by 8-foot sheets of plywood are nailed to two subpurlins; this complete unit is then erected between the larger purlins that are in place. The plywood is then nailed to the purlins, adjacent subpurlins, ledgers, and main roof girders to form a complete diaphragm.

Walls of the masonry wall buildings are commonly of precast reinforced concrete. Wall panels are cast flat, usually on the floor slab and lifted or "tilted" up into place, thus the name "tilt-up buildings." Other types of wall construction include reinforced brick and hollow concrete block. Here, the important factor is interconnecting the units by using good mortar and reinforcing details. This general type of construction is used widely for commercial and industrial buildings in California and many other parts of the country. Some individual structures have floor areas in excess of 1 million square feet. The total investment and the number of people who occupy these buildings are significant. Therefore, the severe damage and collapses that occurred are particularly disturbing. Additional data on this type of construction are included in the paper "Earthquake Damage and Related Statistics."

Another paper, titled "Behavior of Joist Anchors Versus Wood Ledgers," points out the superior behavior of the older buildings of this type where steel joist anchors from the walls to the roof system were used.

Conclusions and recommendations that are common to two or more wood roof and masonry wall buildings are included after the paper on joist anchors and wood ledgers. Conclusions and recommendations considered to be peculiar to an individual building report are included with that report.

The committee feels that the lessons learned in the following reports must be used to determine how design criteria should be improved to prevent collapse and loss of life in the event of the largest credible earthquake expected to occur.

Sylmar Industrial Tract

SUBCOMMITTEE
ON
BUILDINGS

NOAA/EERI Earthquake Investigation Committee

The four buildings studied in this tract are located within a one-block area north of San Fernando Road known as the Sylmar industrial tract (fig. 1). The buildings are Stone's Liquor Store on Bledsoe Street (1) and a Warehouse Building (2) and the All Phase Color building (3), both on Bradley. The Museum for Antique Cars (14) on Bledsoe is also in this tract, but is discussed later since it is a reinforced concrete structure. The site is located approximately 8 miles southeast of the earthquake epicenter and less than 2 miles south of the Olive View Hospital, which was damaged heavily by the earthquake. In this tract, there were numerous similar buildings that suffered similar damage. (See paper titled "Behavior of Joist Anchors Versus Wood Ledgers," this volume.)

The subsurface soils are known to consist of a thick layer of recent alluvial deposits. Soil borings in

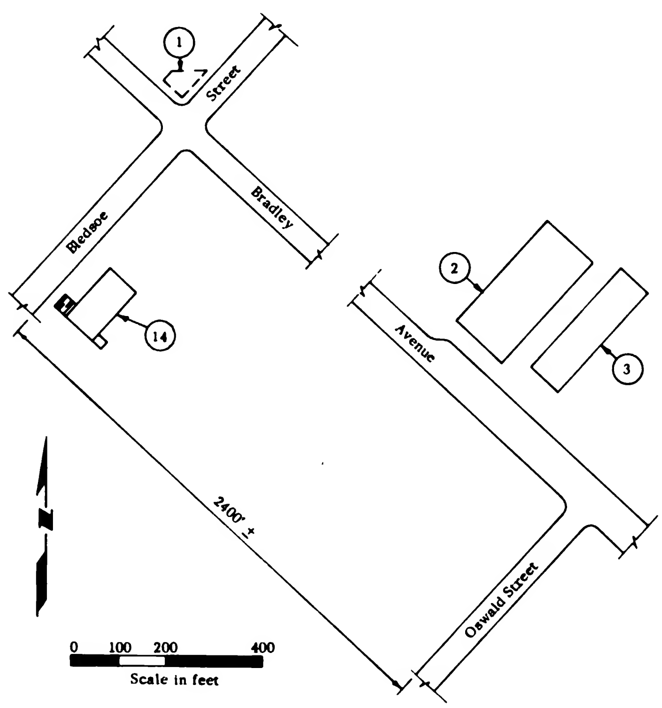

Figure 1.—Sylmar industrial tract. Numbers in circles refer to building report numbers.

the vicinity of the All Phase Color building revealed sand deposits containing moderate-to-large amounts of silt and clay. The natural ground surface has a downward slope of about 3 percent toward the south.

The maximum horizontal ground acceleration at this site is estimated at approximately 40 percent of gravity. No strong-motion instruments were located in the immediate vicinity.

Some permanent ground displacements, generally indicated by ground and pavement cracks, occurred in this area. However, the principal cause of structural damage was the heavy vibration during the earthquake.

Stone's Liquor Store (1)

15151 Bledsoe Street, Sylmar

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Photographs provided by Tom Kamei.

WHEELER & GRAY
Consulting Engineers
Los Angeles, Calif.

GENERAL DESCRIPTION

This building, which was demolished after the earthquake, was trapezoid in plan with bases of 63 and 77 feet (fig. 1). The structure, designed to comply with the provisions of the Los Angeles City Building Code, was built during the latter part of 1969.

It was constructed with a wood roof and enclosed by reinforced hollow concrete block walls approximately 14 feet high from the floor to the top of the roof. The south elevation was mostly openings with glass.

Underlying soils are classified as compact sandy clay with a bearing value of 2,000 psf (pounds per square foot).

Reinforced concrete with a minimum ultimate compressive strength of 2,000 psi (pounds per square inch) at 28 days was used for the footings. A continuous type of footing was used under the walls and isolated spread footings under columns (fig. 1, sections A–A and B–B).

The roof consisted of a plywood system supported by wood rafters, purlins, and glued laminated beams and girders that spanned the building in a north-south direction and cantilevered over the south wall and portions of the west wall. Support of beams and girders was provided by steel pipes, either exposed or enclosed, within wall pilasters (fig. 1).

The plywood used for the roof was ½-inch-thick Douglas Fir, Structural I Grade with exterior glue. The roof nailing consisted of No. 10d plywood nails at 4 inches on center at the boundaries, 6 inches on center at plywood edges, and 12 inches on center at intermediate supports. The plywood roof was designed to perform as a horizontal diaphragm under the influence of wind or seismic loads. The diaphragm carried horizontal forces to the masonry walls. Transfer of horizontal loads from roof to walls was achieved by nailing plywood sheathing to 4-

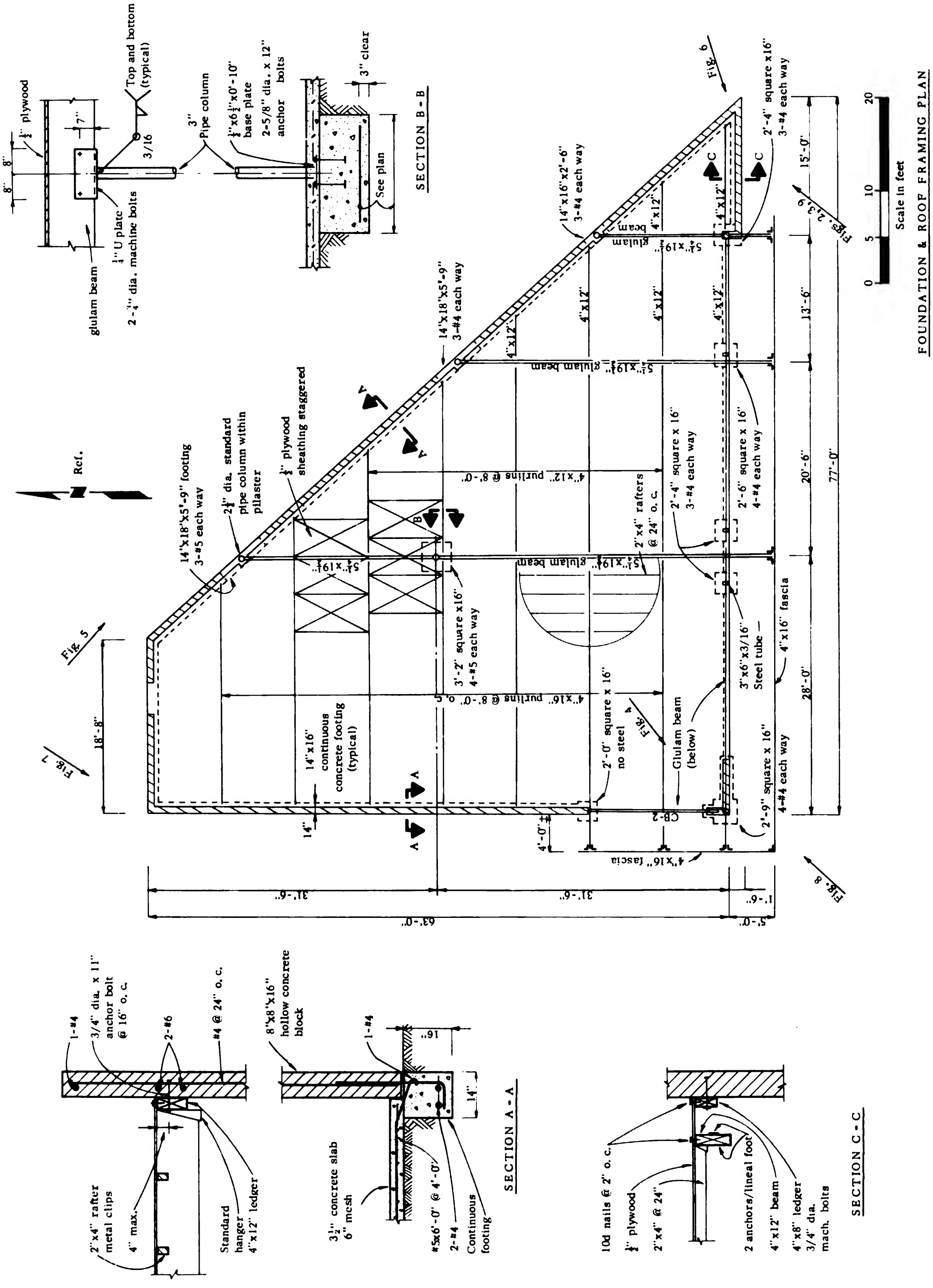

Figure 1.—Stone's Liquor Store. Foundation, roof framing plan, and details.

Figure 2.—Stone's Liquor Store. Concrete block wall at southeast corner of building.

Figure 4.—Stone's Liquor Store. Remains of short shear wall located at southwest corner of building.

Figure 3.—Stone's Liquor Store. Southeast corner of building. Bent steel pipe column can be seen to the left of collapsed pier.

inch-thick ledgers bolted to the wall (fig. 1, section A–A). The south wall contained two short masonry piers that acted as vertical shear-resisting elements. Most of the seismic load to these piers was transmitted axially by 4- by 12-inch wood beams functioning as drag struts or by ties that delivered the loads to the wall through the plywood and ledger (fig. 1, section C–C).

The specified strength of the concrete block masonry is not known. The building appeared to have been designed for the lateral forces required by code.

Figure 5.—Stone's Liquor Store. East wall of building. Apparently, an adjacent and much shorter block fence wall was knocked down by taller building wall.

Figure 6.—Stone's Liquor Store. Southeast corner of the building looking from east.

Figure 7.—Stone's Liquor Store. North concrete block wall severely cracked.

EARTHQUAKE DAMAGE

At the time of the field inspection trip, this building had been demolished completely and removed. As a consequence, evaluation of the damage to this structure is based almost completely on damage indicated in photographs taken shortly after the earthquake, and on conversations with people that observed the building damage. Wall failures were the predominant damage to this structure. The south shear walls completely collapsed (figs. 2, 3, and 4). The east wall was out of plumb and distorted (fig. 5) with severe cracking occurring near the south exit corner (fig. 6). The short north wall also was damaged severely by cracking (fig. 7). Undoubtedly, the existence of pipe columns within the walls prevented the collapse of the roof. Some of these pipes were bent badly but still managed to support their vertical loads (figs. 8 and 9). Only minor floor slab cracks, about 1 inch wide, of a tension type were visible. No vertical displacements were apparent.

This building was a total loss. Its estimated value prior to the earthquake was $35,000.

Figure 8.—Stone's Liquor Store. Southwest corner of building. Collapsed shear wall and bent steel pipe column enclosed within wall.

Figure 9.—Stone's Liquor Store. Collapsed shear wall at southeast corner of building.

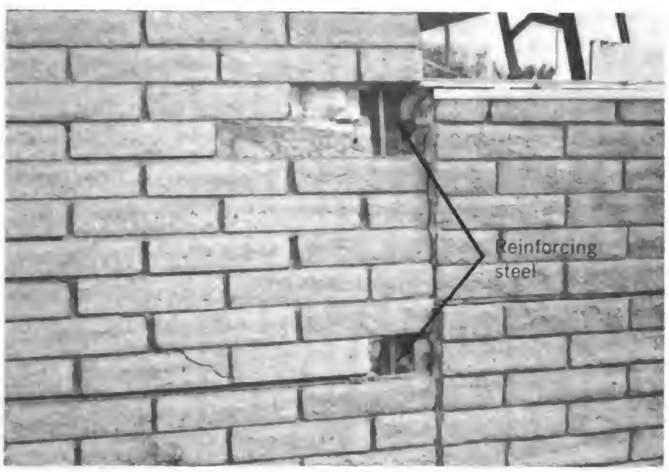

Figure 10.—Stone's Liquor Store. Cell with reinforcing steel does not appear to contain grout.

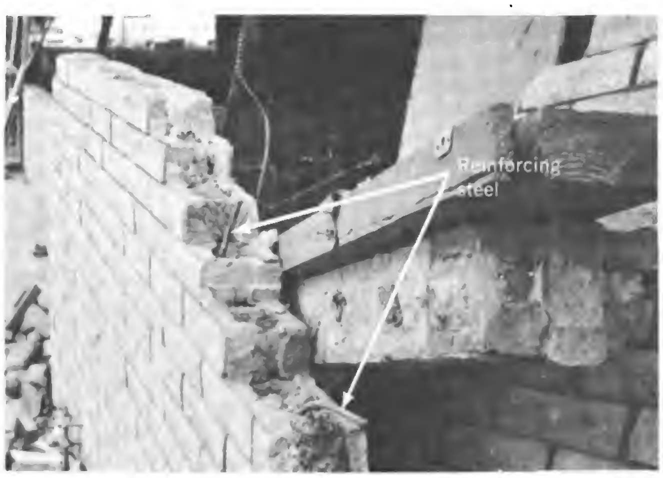

Figure 11.—Stone's Liquor Store. Collapsed full-height wall shows upper end of reinforcing steel. Lack of grout in cell with reinforcing is also apparent.

CONCLUSIONS

The failures of the walls appear to have been the result of one or both of the following actions: (a) When the walls were subject to accelerations perpendicular to their planes, the connections between plywood roof and wall yielded; (b) in the case of the short shear walls in the south elevation, the earthquake-induced forces in the plane of the wall caused the wall to fail in shear. Using forces prescribed by code, it was found that the south wall would have been stressed in shear to approximately 25 psi. The allowable shear stress is 12 psi with an increase of 33 percent. For a properly built wall of this type, an ultimate strength in shear of 40 to 50 psi could be expected.

Figures 10 (west wall) and 11 (south wall) show vertical cells with reinforcing, but without grout. Observation of figures 9 and 11 indicates that vertical steel may not have extended over 5 or 6 feet above the floor, which leaves portions of the wall unreinforced. An apparent lack of adequate steel continuity around corners is indicated in figures 2 and 8.

RECOMMENDATIONS

1 Full continuity of horizontal wall steel should be developed at wall corners and intersections.

2 The practice of concentrating horizontal wall reinforcing at various levels should be reevaluated.

3 Continuous masonry inspection should be required. Although building department records indicate 20 inspections during construction, there were cases of missing grout and reinforcing.

Warehouse Building (2)

12884 Bradley Avenue, Sylmar

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WHEELER & GRAY
Consulting Engineers
Los Angeles, Calif.

GENERAL DESCRIPTION

This one-story building was designed to comply with the 1968 edition of the Los Angeles City Building Code. By the date indicated on the drawings, it is estimated that actual construction must have taken place during 1970. The building either was just completed or was in the last stages of completion when the earthquake struck.

The general notes on the drawings indicate that a soil investigation was conducted. This report was not available, but from adjacent construction it is concluded that underlying soils are of a sandy clay type. Eighteen-inch-diameter reinforced concrete, drilled and poured-in-place piles with lengths ranging from 15 to 20 feet were used for foundations (foundation plan, fig. 1, sections E–E and F–F).

This building is rectangular in shape, 131 feet 6 inches by 276 feet, and consists basically of a plywood roof system on wood subpurlins and glued laminated wood girders spanning across in a north-south direction (fig. 1).

The south girders span from a corbel provided on the south wall to a steel pipe column located at the center of the building. For economy of design, the girders cantilever 12 feet past the column to pick up the second and shorter girder (fig. 1, detail 3) whose other end rests on a similar north wall corbel. Connection of the glued laminated girder to the corbels is accomplished by the use of a commercially available beam seat anchor made out of steel plates and bolted to the girders and to the concrete (fig. 1, section C–C).

The exterior walls are precast concrete panels erected by the "tilt-up" procedure, and interconnected by poured-in-place concrete closures. The longitudinal walls (north and south) are almost solid with only two openings at the south side, while the front and back (east and west) are mostly open with only two solid panels at each elevation.

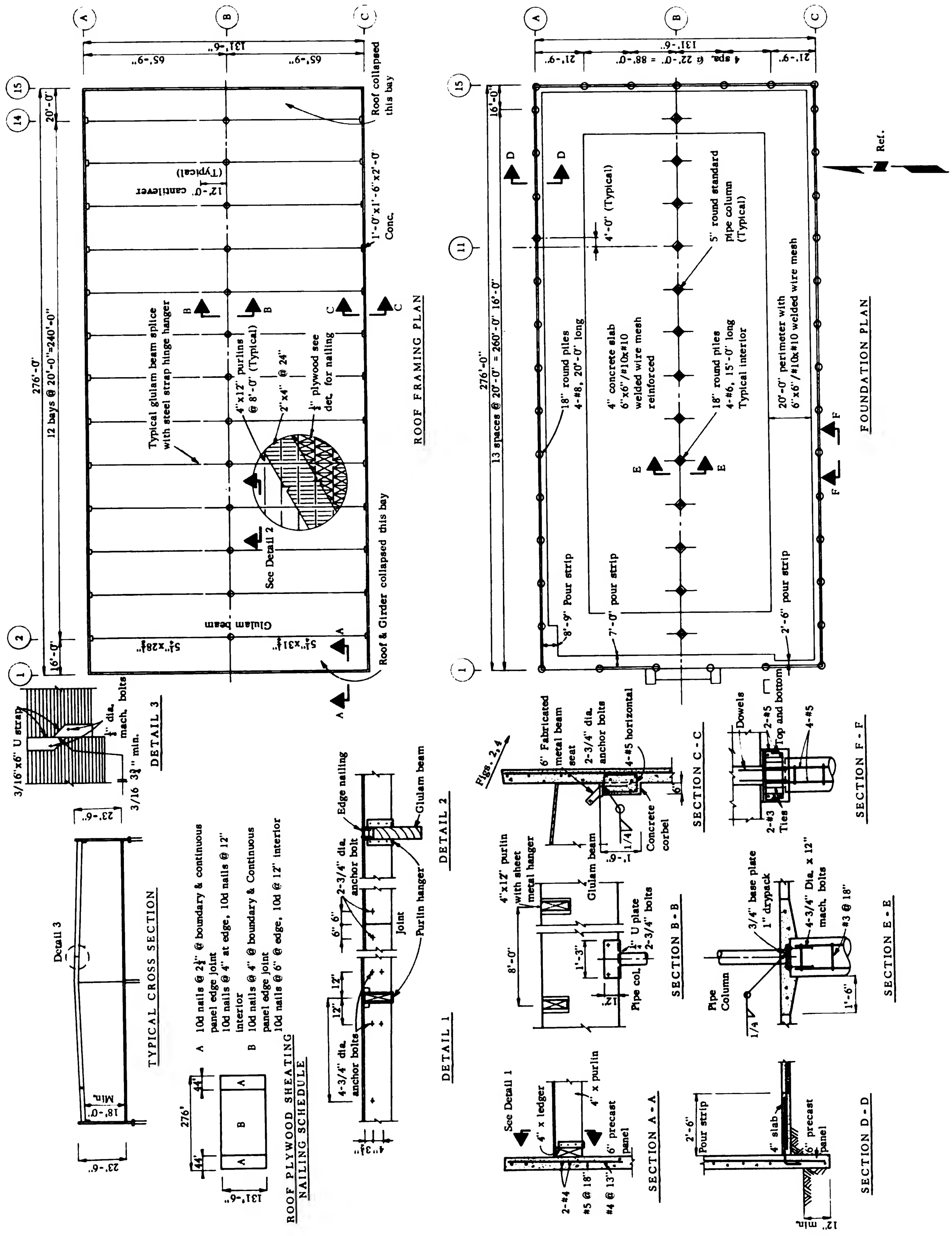

Figure 1.—Warehouse Building. Roof framing plan and foundation plan.

All the concrete used had a specified ultimate compressive strength of 2,000 psi at 28 days with the exception of the piles that required a concrete strength of 2,500 psi.

When this structure is subjected to wind or seismic forces in either direction, the ½-inch plywood roof is designed to work as a horizontal diaphragm, transferring the forces to the exterior of the roof sheathing to a 4-inch-thick wood ledger bolted to the wall (fig. 1, section A–A).

In general, this structure appears to have been designed in accordance with the requirements of the Los Angeles City Building Code.

EARTHQUAKE DAMAGE

Damage suffered by this structure was basically of the same nature as the damage sustained by numerous buildings of this type throughout this area. Permanent ground disturbances were not apparent; only small floor cracking could be observed.

At the time of the field inspection trip most of the repairs were already finished, and the evaluation of the damage had to depend on the information received from the working crews, damage report prepared by the Los Angeles city inspector, available photographs, and repairs made that were still detectable.

Portions of the roof collapsed at the end bays of the building (figs. 1 and 2). The large horizontal forces induced by the seismic accelerations exceeded the capacity of the roof-to-wall connection. The supporting precast concrete walls and beams separated from the roof when the nailing attaching plywood to ledger pulled through the edges of the plywood and, in some instances, the wood ledger bolted to the wall split at the bolt line. The walls, although out of plumb and bowing out, managed to remain in place except at the northwest corner. Wood purlins supporting the roof in the end bays lost their bearing, carrying the roof down with them. A concrete column at the front of the building was cracked severely.

Glued laminated girders on line 2 (fig. 1) fell with the roof. Wall corbels supporting laminated girders were cracked, and the connection between both elements became ineffective. Several girders sustained either longitudinal cracks or partial separations of laminations.

A large percentage of glass windows at the front of the building were broken; frames and mullions jammed and buckled (fig. 2).

Figure 2.—Warehouse Building. Concrete beams and column framing at northwest corner of the building collapsed. Note broken glass in entrance area. Los Angeles City Department of Building and Safety photograph.

Figure 3.—Warehouse Building. Glued laminated girder with gusset plate where split or delamination occurred. Repaired corbel on wall. Wheeler & Gray photograph.

REPAIRS

Concrete walls had to be straightened and braced to proper alignment before other corrective work could be started. The roof was reconstructed to original condition, which included replacement of ele-

Figure 4.—Warehouse Building. Reconstruction of reinforced concrete structure at northwest corner of building. Wheeler & Gray photograph.

ments severely damaged. Steel gusset plates were bolted to the glued laminated girders to reinforce the areas where cracking and separations occurred (fig. 3). Reinforced concrete that collapsed at the northwest corner of the building was repoured (fig. 4). Cracks in concrete were pressure injected with epoxy resin compounds, and almost every corbel had to be rebuilt after all of the affected concrete was removed (figs. 3 and 5). It is estimated that the cost of earthquake repairs was about $80,000, which represents about 25 percent of the value of the building.

CONCLUSIONS

The connection between the girders and corbels consistently failed under the level of forces originated by this earthquake.

Figure 5.—Warehouse Building. Wall corbels had to be repoured. Girder rests on temporary shoring. Wheeler & Gray photograph.

RECOMMENDATIONS

1 Improvement of the connections between main girders and corbels is recommended.

2 Design of girders should take into consideration axial forces and secondary moments induced by lateral forces.

All Phase Color (3)

12874 Bradley Avenue, Sylmar

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WHEELER & GRAY
Consulting Engineers
Los Angeles, Calif.

GENERAL DESCRIPTION

This structure is one-story with a small second floor in the office area at the west end (front) of the building (fig. 1). The drawings state that the design of this building complies with the 1967 edition of the Uniform Building Code and/or the Los Angeles City Building Code.

A foundation investigation was performed but was not available. Soils are classified as sandy clay with a maximum bearing value of 1,900 psf used in design of footings.

The roof consists of a ½-inch plywood sheathing, Structural I Grade, supported on wood rafters and purlins, and tapered steel girders spanning across the building in a north-south direction, resting on concrete columns cast between wall panels. The second floor consists of wood joists supported by wood stud and precast concrete walls. An open deck occupies the west 20 feet of the second floor, and the remaining space is used for office and laboratory (fig. 2).

The north, south, and east exterior walls are precast concrete panels of the tilt-up type. The west wall is reinforced brick, and those enclosing the open deck are wood studs. Inside the building, offices are separated from the manufacturing area by a wood stud wall at the south and by a full-height precast reinforced concrete wall at the east (fig. 1). Reinforced concrete block walls in combination with wood stud walls enclose an interior room (fig. 2, section A–A; fig. 3, sections B–B and C–C). The precast concrete walls are supported by reinforced concrete isolated spread footings located under wall columns. Other walls rest on continuous footings (fig. 1, foundation plans).

The concrete specified for this construction had an ultimate compressive strength of 2,000 psi at 28 days. The units used for the brick wall were medium weathering (MW) grade with a minimum ultimate compressive strength of 2,500 psi. Only the cells con-

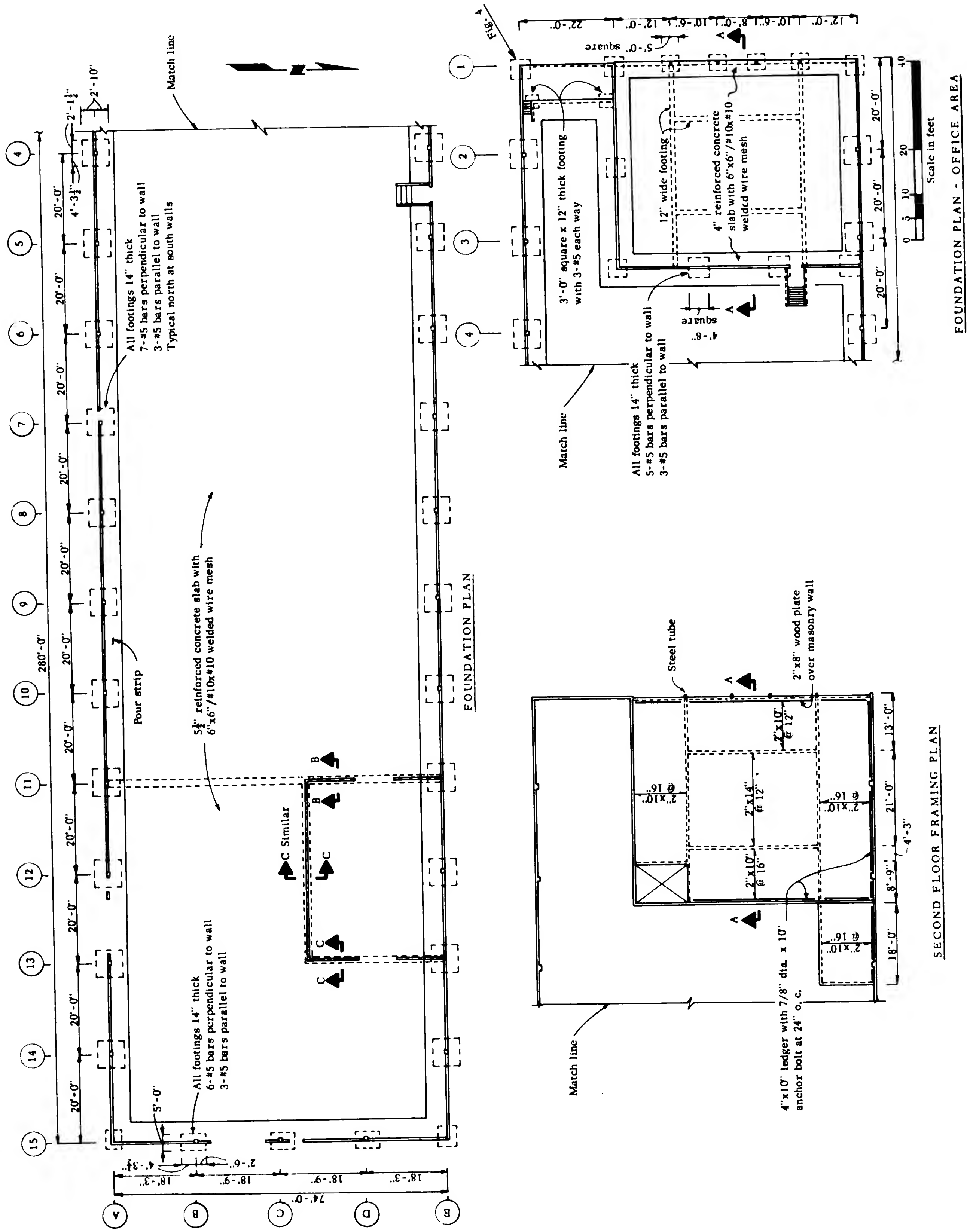

Figure 1.—All Phase Color. Foundation plan, second-floor framing plan, and foundation plan — office area.

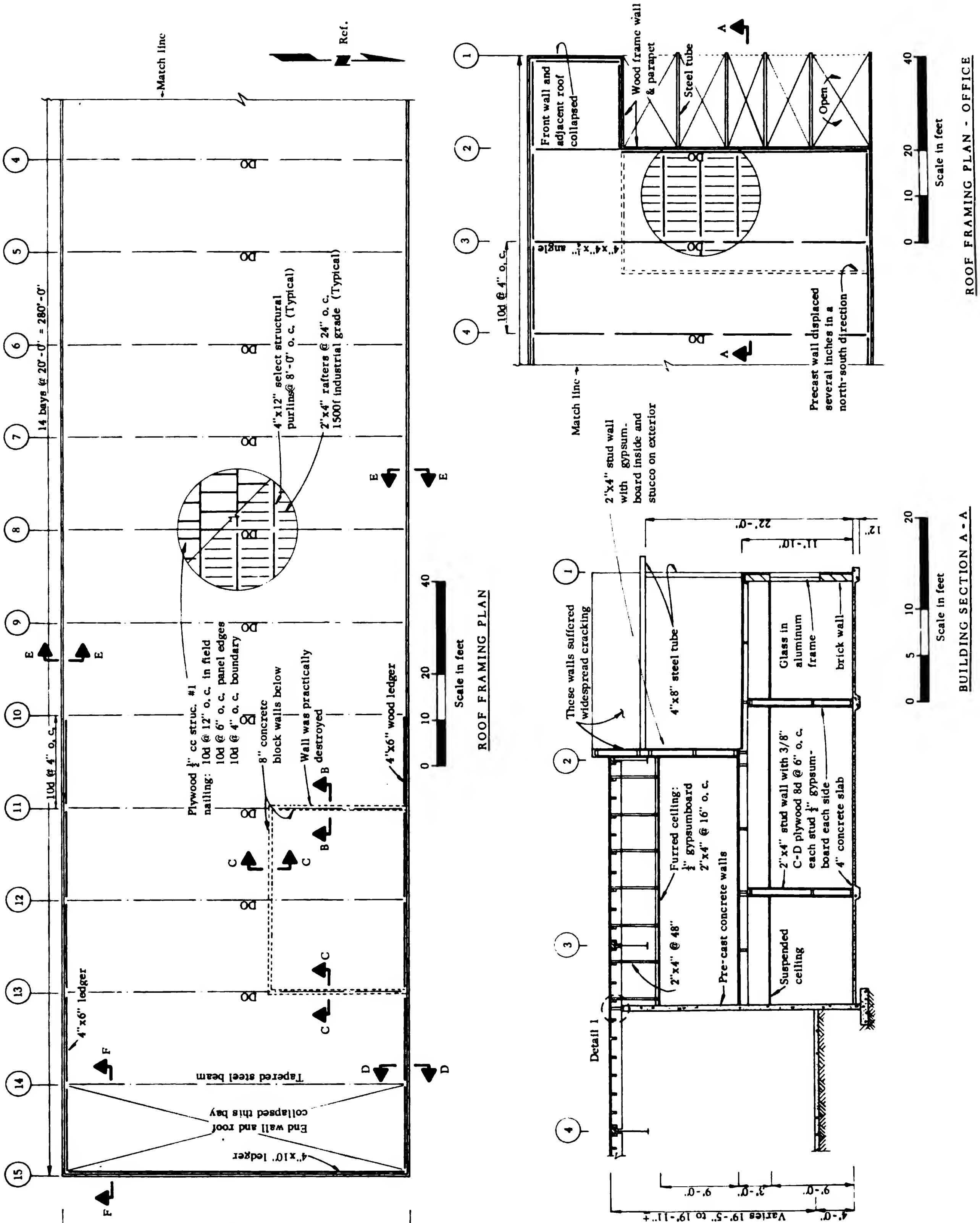

Figure 2.—All Phase Color. Roof framing plan, building section A–A, and roof framing plan —office area.

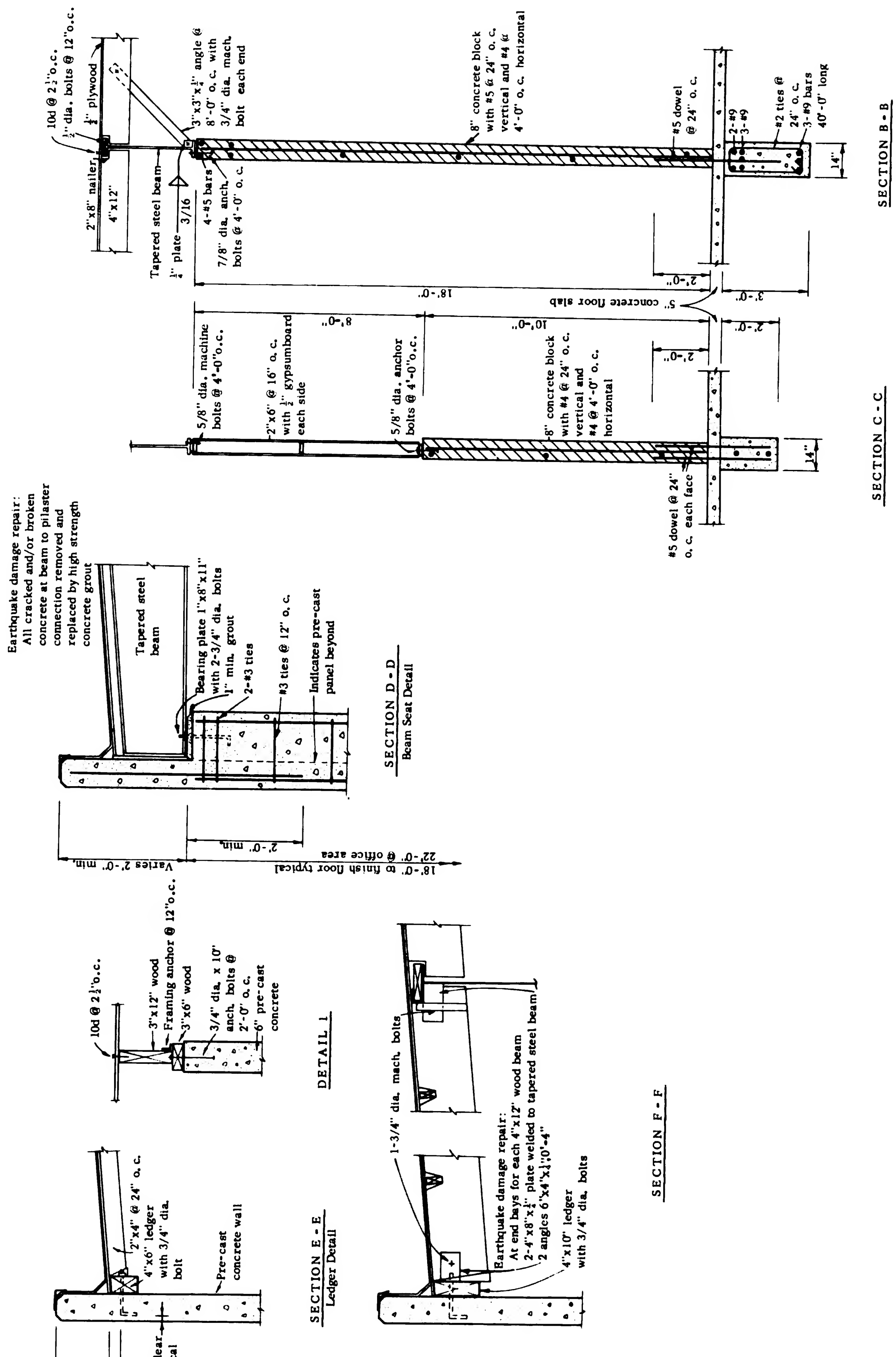

Figure 3.—All Phase Color. Ledger details and wall sections.

taining reinforcing steel were grouted in the concrete block walls, and no reference was found concerning the design strength of the masonry. Since no special inspection was required in the general notes, it is assumed that masonry was designed for half the allowable stresses as provided for by code.

For lateral forces, the plywood roof sheathing functions as a horizontal diaphragm, distributing and transferring horizontal forces to the walls connected to it. In the east-west direction, exterior concrete precast walls provide ample lateral support. For north-south requirements, the intermediate precast concrete wall (fig. 1, line 3.3) and the concrete block wall (fig. 1, line 11) were connected to the roof to reduce the span of the roof diaphragm. Roof-to-wall connections are shown in figure 3, detail 1 and sections B–B and C–C.

The connection between the plywood diaphragm and the exterior concrete precast panels is developed through a 4-inch-thick wood ledger bolted to the wall (fig. 3, sections E–E and F–F). Roof sheathing is nailed to the ledger with 10d nails (3 inches long).

The lateral design of this building appears to comply with the 13.3 percent of dead load required by code.

Figure 4.—All Phase Color. West elevation of building. Precast concrete panel and adjacent roof collapsed. Los Angeles City Department of Building and Safety photograph.

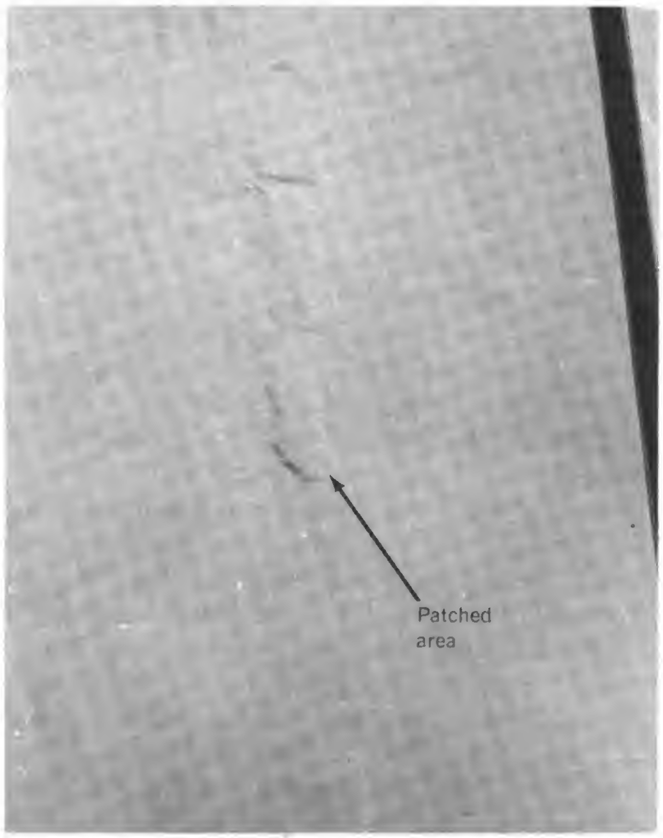

Figure 5.—All Phase Color. North wall, looking from outside the building. Though repaired, damage inflicted by displacement of interior wall is evident. Wheeler & Gray photograph.

EARTHQUAKE DAMAGE

Some ground disturbance took place in this area. Cracks of a tension type up to ½ inch wide appeared through the concrete slab at random locations. Vertical differential displacement was not visible, and there was no evidence of foundation failure. At the time of the field inspection the building was already repaired. However, a building damage report by Los Angeles city, photographs, damage repair drawings, and descriptions by occupants of the building clearly indicated the nature and extent of damage caused by the earthquake.

The east wall of the building and adjacent roof collapsed. The high west façade wall at the loading dock and the immediate roof supported by the wall also collapsed (fig. 4). It can be assumed safely that the roof sheathing and/or wood ledger separated from the wall at these areas. The walls and roof purlins lost their horizontal and vertical supports, respectively, with collapse of the wall and roof following.

Ground movements displaced the transverse precast shear wall on line 3.3 (fig. 2) approximately 4 inches in a north-south direction, inflicting severe damage to the exterior precast panels connected to this wall (fig. 5). Most exterior concrete walls exhibited cracking in random directions.

The concrete block wall at line 11 (fig. 1) was damaged badly and separated from exterior panels. It has been determined that the shear in this block wall from code forces would be approximately 38 psi.

The exterior stud wall over the second floor at line 2 (fig. 2) was fractured, apparently when its shear capacity was exceeded. The tapered steel girders are anchored to the wall pilasters by means of a bearing plate and 3/4-inch anchor bolts. Forces transferred through this connection were sufficiently large to fracture and spall the upper part of most pilasters (fig. 2, detail 1).

REPAIRS

It is understood that the building was rebuilt to preearthquake condition. No changes in the original design were made except those described in the following paragraphs. All affected walls and roof areas were repaired when their condition permitted; otherwise, they were removed and replaced.

New clip angles connecting end walls to purlins were installed. Also, new steel gusset plates were welded to the tapered steel girders and bolted to the purlins at each end bay (fig. 3, section F–F).

The two concrete panels forming the southwest corner were replaced by steel stud walls except the lower 4 feet that remained concrete. All cracks in slab and concrete wall panels were pressure injected with epoxy resin material. Walls that were out of plumb and misaligned were straightened to the original position. Additional horizontal steel rod bracings were provided at one bay of the high roof, directly over the loading dock. The repair costs are estimated to be about $70,000, approximately 25 percent of the building value prior to damage.

RECOMMENDATIONS

Special attention should be given to continuity of steel and ties at wall intersections.

San Fernando Industrial Tract

SUBCOMMITTEE ON BUILDINGS

NOAA/EERI Earthquake Investigation Committee

The four buildings studied in this section are located within close proximity to each other in an area known as the San Fernando industrial tract (fig. 1). There were numerous buildings of similar construction in this tract, most of which suffered similar damage. The epicenter of the earthquake was located about 8 miles north of this tract. Approximately 2¼ miles due north of this site is the Veterans Administration Hospital (44). Located 1 mile south of the site is the Foothill Medical Center (18), which was damaged extensively. The four buildings studied in this area are Bell Metrics on West Arroyo, Sylmar (4); M&L Machine Shop (5) and Vector Electronics Company (6) on Gladstone, Sylmar; and Wendell Machine Shop (7), Foothill Boulevard, Sylmar. All of the buildings are of the one-story industrial type.

Subsurface soils are known to be recent alluvial deposits generally made up of a sand-gravel mixture. The area has a gentle 2-percent slope down toward the southwest.

Although there were no recorded levels of ground motion at the site, it is estimated that the maximum horizontal ground acceleration was approximately 40 percent of gravity. A considerable amount of permanent ground movement and surface fault breakage occurred in this area. Some permanent differential horizontal and vertical ground movements occurred. These permanent ground movements undoubtedly had a significant influence on structural behavior of some of the buildings in this tract. However, vibratory movements during the earthquake were the major cause of damage.

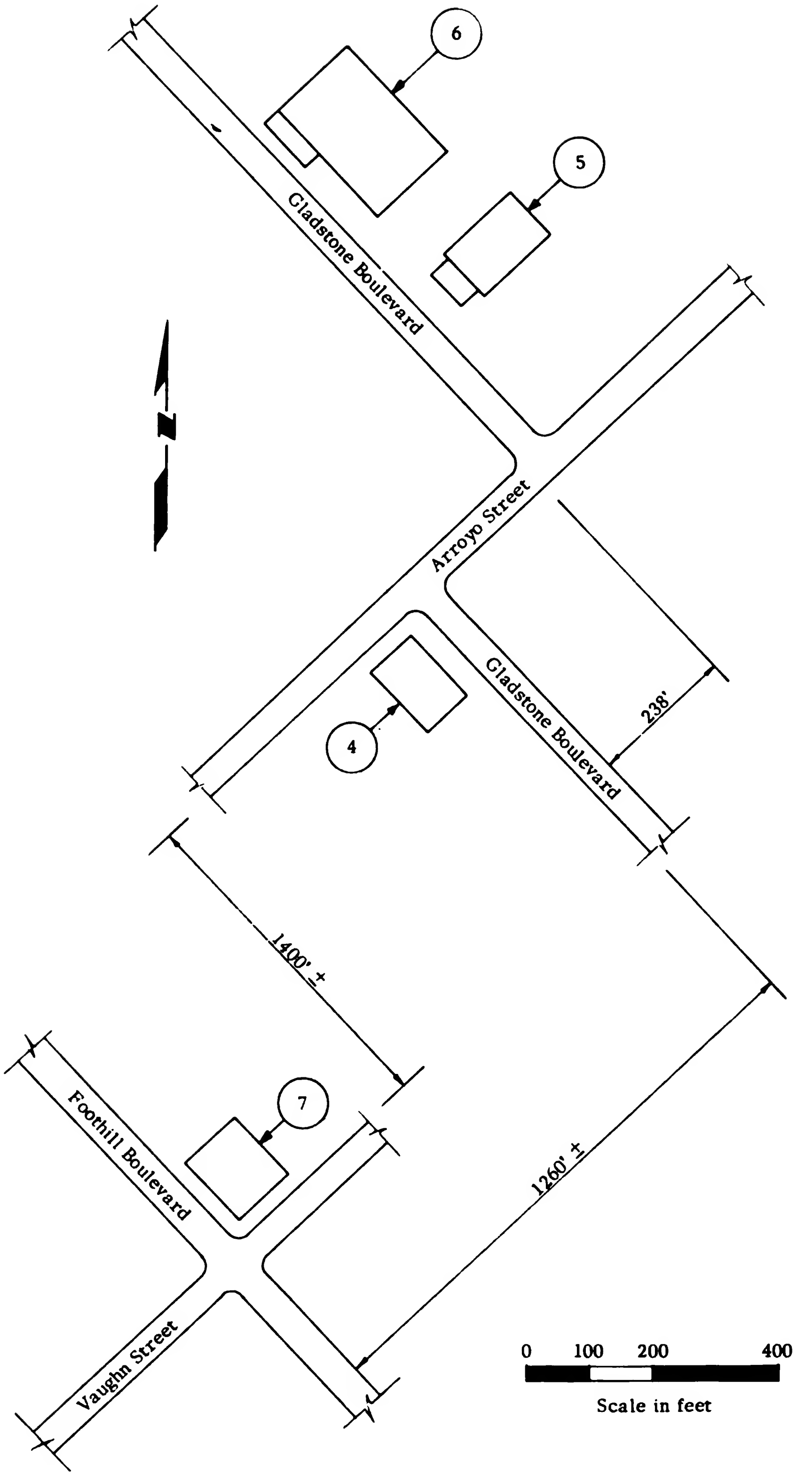

Figure 1.—San Fernando industrial tract. Numbers in circles refer to building report numbers.

Bell Metrics (4)

12836 West Arroyo Avenue, Sylmar

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WHEELER & GRAY
Consulting Engineers
Los Angeles, Calif.

GENERAL DESCRIPTION

This building is rectangular in plan measuring 96 by 137 feet. It has an all-wood roof enclosed on four sides by reinforced brick walls (fig. 1). The structure is founded on alluvial soils, classified as compact clayey sand. A design value of 1,500 psf was used for foundations. A soils investigation report was not available.

The front (north) portion of the building is occupied by offices and the rest is a manufacturing area.

The roof is ⅜-inch plywood, Douglas Fir, with face grain perpendicular to rafters. Nails used were 8d (2½ in. long). Wood rafters and purlins are supported by glued laminated wood girders that span 48 feet in an east-west direction from the reinforced brick pilasters built with the wall to the center steel columns (fig. 2, sections A–A and D–D).

The foundations consist of continuous reinforced concrete footings under the exterior walls and reinforced concrete isolated spread footings under the interior columns (fig. 2, sections A–A and D–D). A 4-inch-thick concrete slab on grade, reinforced with welded wire mesh, forms the floor. Drawings indicate that the concrete used in construction had an ultimate compressive strength of 2,000 psi at 28 days. When the building is subjected to horizontal forces, the plywood roof functions as a horizontal diaphragm, carrying and distributing the lateral forces to shear walls. The exterior reinforced brick walls act as shear walls in both directions. Two No. 5 bars in the wall, located at roof level, help to distribute horizontal forces and also to resist tensile chord stresses induced in the wall by the diaphragm (fig. 2, sections A–A and B–B). The roof sheathing is attached to the wall by nailing the plywood to 4-inch-thick wood ledgers that are bolted to the wall (fig. 2, section B–B). The ledgers also support the roof rafters and purlins by means of metal hangers.

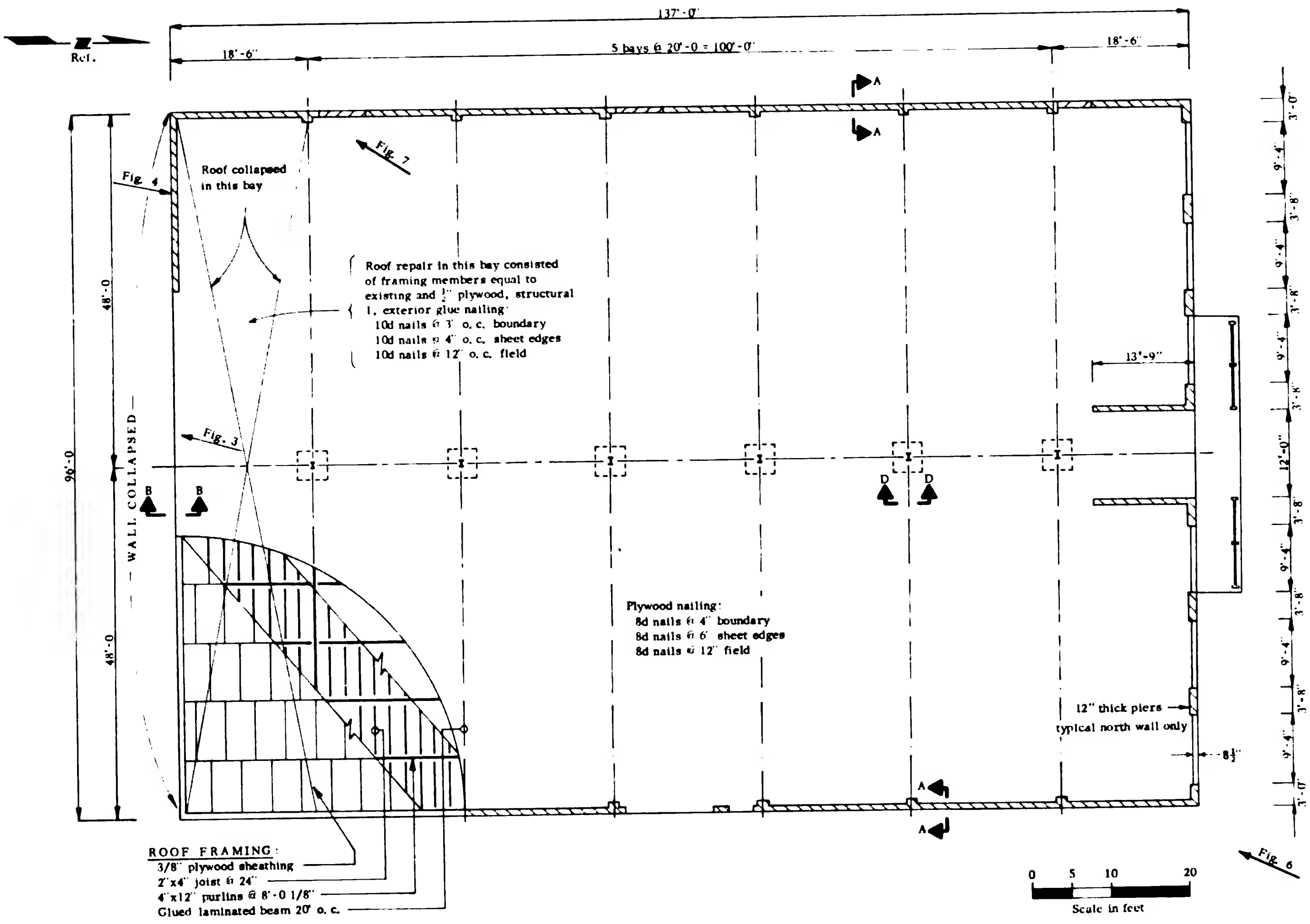

Figure 1.—Bell Metrics. Roof framing plan and floor plan.

Glued laminated girders are anchored to the masonry pilasters through a steel base plate with anchor bolts. A vertical plate welded to the base plate is bolted to the girder (fig. 2, section A–A). Brick units used in the walls are 2,500 psi. The structure was designed to conform to the provisions of the Los Angeles City Building Code, which specified a seismic factor of 13.3 percent of dead loads. Code requirements appear to have been met.

EARTHQUAKE DAMAGE

At the time of the field trip all repairs were made. As a source of information, photographs, reports, and personal descriptions were used.

No major soil disturbance seemed to have affected the immediate area where this building was located. Only some minor ground cracking was visible. The rear (south) wall of the building collapsed after losing lateral support at the roof (figs. 3 and 4). The nailing connecting the plywood roof to the ledger ripped through the edge of the plywood. Even though the ledgers did not fail, some of them showed serious splitting (fig. 5). The front wall of the building also was loosened from the roof but did not come down. Apparently, the wall returns at the entrance lobby helped to prevent the wall from collapsing. One of the side walls (west) moved out several inches at the front two bays, and the bolts connecting the ledgers to the wall pulled loose at the first bay only.

At the northeast corner of the building, walls separated, bricks from the upper portion broke and fell, and a pronounced crack appeared in the middle height of the wall (fig. 6). Anchorage between glued laminated girders and masonry pilaster was lost when the upper 2 feet of masonry developed severe cracking and, in some cases, spalled off (fig. 7).

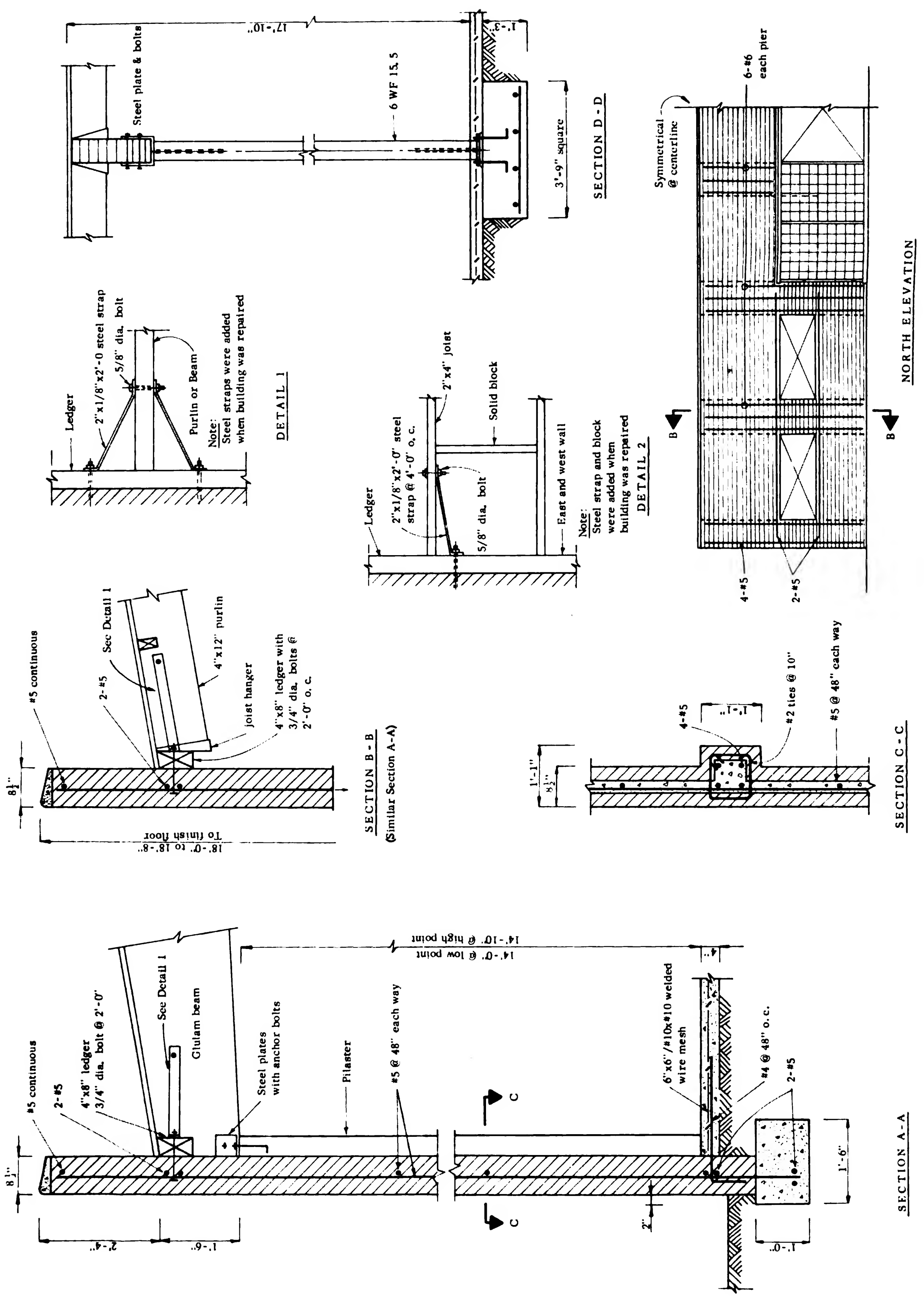

Figure 2.—Bell Metrics. Wall sections and details.

Figure 3.—Bell Metrics. South wall of building on ground after collapse. Wood ledger still attached to wall in upper part of photograph. J. F. Meehan photograph.

Figure 4.—Bell Metrics. South wall collapsed. Note vertical reinforcing torn from brick wall. J. F. Meehan photograph.

Figure 5.—Bell Metrics. Closeup of wood ledger still attached to wall after collapse. Note nails in ledger. J. F. Meehan photograph.

Figure 6.—Bell Metrics. Northeast corner. Note separation. Bell Metrics photograph.

Figure 7.—Bell Metrics. West wall, with top of brick pilaster spalled off. Temporary repairs showing under girder. Bell Metrics photograph.

REPAIRS

This structure was rebuilt to preearthquake conditions including some minor changes that were requested by the Los Angeles City Building Department. One of them was the addition of steel straps bracing from rebuilt walls to roof members (fig. 2, details 1 and 2). The other was the use of ½-inch plywood with heavier nailing for that section of roof that collapsed (fig. 1).

It is estimated that the cost of earthquake repairs was about $33,000 or 25 percent of the building value prior to damage.

RECOMMENDATIONS

Special attention should be given to continuity of reinforcing steel at corners of walls.

M&L Machine Shop (5)

12424 Gladstone Avenue, Sylmar

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WHEELER & GRAY
Consulting Engineers
Los Angeles, Calif.

GENERAL DESCRIPTION

This one-story structure, built in 1969, consists of a large shop area and an adjoining office. The original ground surface slopes up toward the northeast at 2 to 3 percent. The underlying alluvial soils are compact silty sand. Design foundation bearing values were 1,000 psf with a one-third increase for lateral loads. Neither soils boring data nor the foundation report was available for this site. However, this building is located about 100 feet south of Vector Electronics Company (6), discussed in the following report, and presumably, the soil conditions would be similar for the two structures.

The wood frame office area, 37 by 67 feet in plan, is attached to the west side of the shop area (fig. 1). The shop is 90 by 142 feet in plan. It has a plywood roof system supported on sawed wood beams and glued laminated wood girders spanning in the north-south direction. Shop walls are 8-inch-thick reinforced concrete block bearing type with pilasters along the north and south walls only.

There is a center row of wood columns extending in the east-west direction in the shop area. The glued laminated girders in the south portion of the area span 45 feet from the south exterior masonry bearing wall over the center row of columns, and cantilever 8 feet 6 inches beyond to pick up the shorter girders in the north portion (fig. 1, section B–B).

The office area roof framing rests on the west reinforced concrete block wall of the shop.

Foundations are reinforced concrete spread type. Continuous footings are used under masonry and wood bearing walls. Isolated spread footings support interior columns. The 4-inch-thick floor slab is reinforced with welded wire mesh and rests on grade.

The lateral force-resisting system for the office consists of ½-inch-thick plywood roof sheathing that acts as a horizontal diaphragm to distribute forces to the

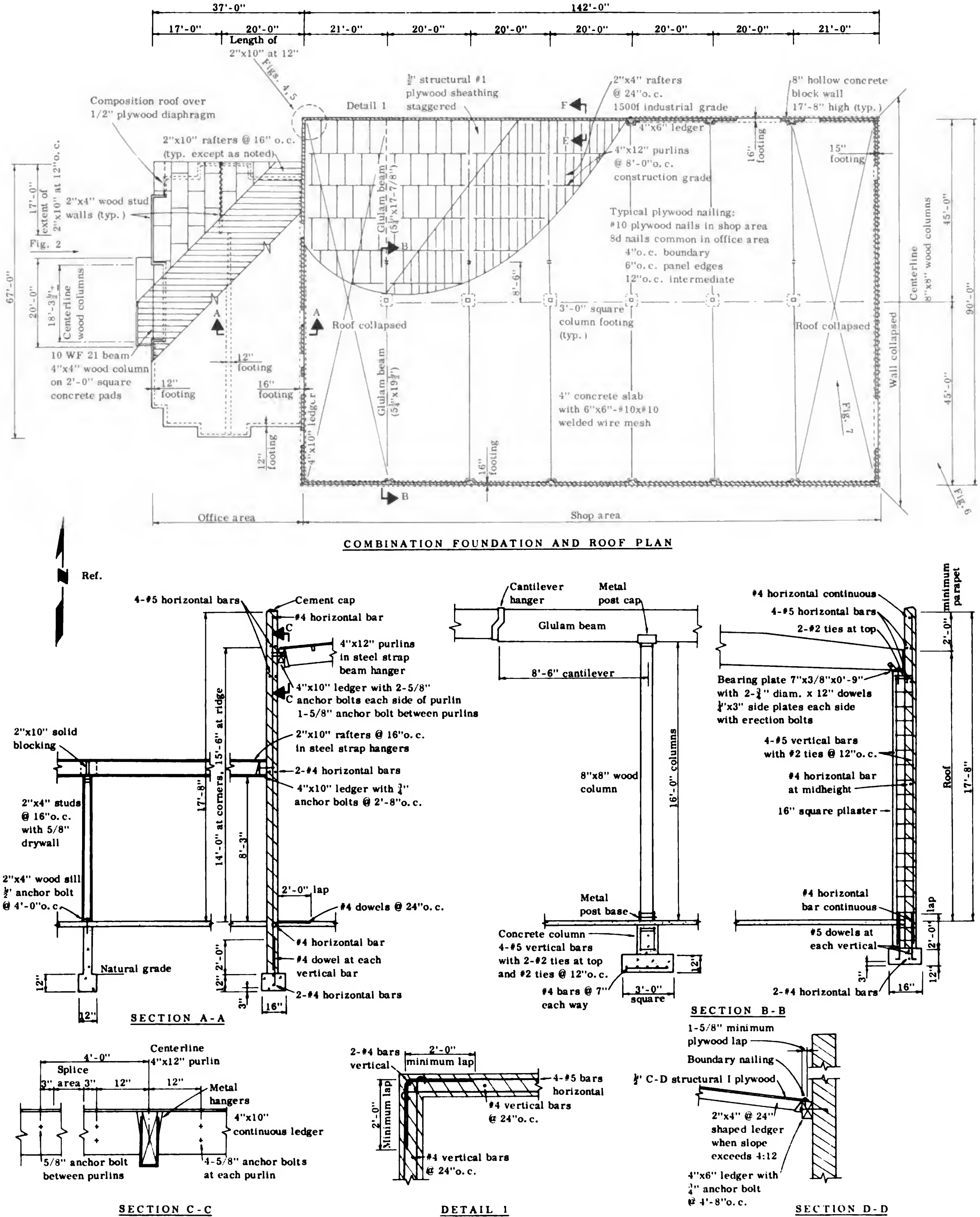

Figure 1.—M&L Machine Shop. Combination foundation and roof plan.

wood stud shear walls, which are sheathed with 5/8-inch-thick gypsum wallboard (fig. 1, section A–A). The west masonry wall of the shop area also acts as a shear wall for the office area.

One-half-inch-thick plywood sheathing also acts as a horizontal diaphragm for the shop area. The exterior reinforced masonry walls act as shear walls in both directions, and have four No. 5 horizontal reinforcing bars, located near the roofline, which resist chord tensile stresses developed by the diaphragm (fig. 1, detail 1).

In the shop area, the roof sheathing is attached to the wall by nailing the plywood to 4-inch-thick horizontal wood ledgers that are bolted to the masonry walls (fig. 1, sections C–C and D–D). Wood purlins and rafters are supported in metal hangers nailed to the 4-inch-thick wood ledgers.

The glued laminated girders are anchored to the pilasters by a metal seat device with a 1/2-inch bolt through the ends of the girders (fig. 1, section B–B).

The Los Angeles City Building Code, which required a lateral force factor of 13.3 percent of dead load, was used in designing the building. Ultimate concrete compressive design strength was 2,000 psi at 28 days.

Concrete block units were the hollow lightweight type with vertical reinforcing bars placed in vertical cells. Only cells containing reinforcing were grouted. The masonry evidently was designed using low stresses, indicating that continuous inspection by a registered deputy inspector was not required during construction.

Plywood roof sheathing was Douglas Fir, Structural I Grade. Overall lateral force design appears to conform with code requirements.

EARTHQUAKE DAMAGE

This general area was disturbed by surface effects of ground faulting. At this particular site the soil disturbance consisted of several tension-type cracks running roughly in an east-west direction (fig. 2). In some cases the building floor slab was cracked in the same manner (fig. 3).

Differential vertical ground movements could not be detected. Evidence of foundation failure was not noted.

At the time of the inspection (June 1971), all building repairs had been completed except for some patching of floor slab cracks. The principal sources

Figure 2.—M&L Machine Shop. Front of building. Typical tension crack through street. Wheeler & Gray photograph.

Figure 3.—M&L Machine Shop. Floor slab crack through office area. Wheeler & Gray photograph.

of damage information were reports prepared by the Los Angeles Building Department engineers, photographs, and conversations with the owner. Wall corner damage and corrective repairs are shown in figures 4 and 5.

Heaviest damage occurred in the shop area where the entire east wall and portions of the north and south masonry walls completely collapsed (fig. 6). Portions of the roof framing adjacent to the east wall, as well as the last bay adjacent to the west wall (fig. 7), collapsed.

There was considerable evidence of failure of the roof plywood-to-wall connection. Failure consisted mostly of plywood nails pulling through or laterally out of the sheets. In a few cases nails pulled from ledgers, and some ledgers split horizontally along the upper line of bolts. Several sheets of plywood remained attached to the ledger, but nailing failed at the first joint inward from the wall.

Failure of the plywood-to-ledger connection permitted the purlins to pull out of their seats and to fall to the floor. Wall collapse followed the roof collapse, since the wall would be required to act as a vertical cantilever above the floor slab and could have been subjected to pounding from the roof system before failure.

Figure 5.—M&L Machine Shop. Walls separated several inches at this corner. Walls were partially rebuilt (see fig. 4). Wheeler & Gray photograph.

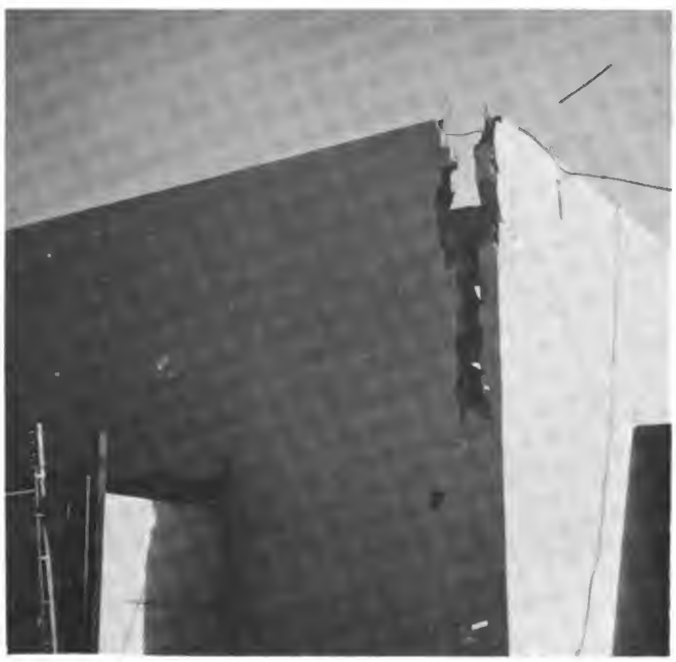

Figure 4.—M&L Machine Shop. Northwest corner. D. F. Moran photograph.

Figure 6.—M&L Machine Shop. Rear (east) collapse. D. F. Moran photograph.

Figure 7.—M&L Machine Shop, looking north. Collapsed roof area. D. F. Moran photograph.

At the north and south walls, the tops of some of the masonry pilasters, which support the glued laminated girders, were cracked and spalled.

Nonstructural damage to ceilings and wall finishes occurred in the office area.

REPAIRS

Drawings covering the earthquake repairs were not available for review, but a visual inspection indicates that the structure essentially was restored to its original condition.

Damaged masonry walls were removed to undamaged areas and rebuilt to their original conditions. Tops of masonry pilasters were repaired where they were cracked and spalled.

In the areas where the roof was damaged or separated from the masonry walls, joist anchors or steel straps were added to anchor the masonry walls to the roof.

It is estimated that the cost of earthquake repairs was about 20 percent of the total value of the building prior to damage.

CONCLUSIONS

Wall corner separations could be the result of a concentration of horizontal reinforcing at the tops of walls, rather than a more even distribution throughout the full height. Corner damage probably was caused by movements of the roof diaphragm parallel to the shear walls, as well as by excessive diaphragm deflections.

RECOMMENDATIONS

Continuity of wall construction at corners should be improved.

Vector Electronics (6)

12460 Gladstone Avenue, Sylmar

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WHEELER & GRAY
Consulting Engineers
Los Angeles, Calif.

GENERAL DESCRIPTION

The Vector Electronics building is a 40,000 square-foot one-story structure with tilt-up concrete walls and a plywood roof supported on wood purlins and glued laminated beams (figs. 1 and 2). The building was completed in 1968.

A detailed soils investigation was not made on the site prior to construction. Underlying soils are a well-graded, gravel-sand mixture with some fines that make up alluvial sediments, derived from past outflow, principally from the Pacoima and Limekiln Canyons. Footing pads supporting concrete pilasters and interior steel columns were designed for a maximum allowable soil bearing pressure of 3,400 and 3,000 psf, respectively. A soils investigation of the site was made subsequent to the February 9 earthquake to prepare remedial plans. This investigation concluded that the existing foundations were satisfactory for continued support of the repaired structure.

The northwest corner of the building has a low roof over the offices (fig. 2; fig. 4, building section). This area adjoins the high roof over the major portion of the building. The floor is typically slab on grade with a predominant thickness of 4 inches. Slab reinforcing is by wire mesh. Precast concrete wall panels, erected by tilt-up procedures, have a thickness of 6 to 7½ inches (fig. 3, sections D–D, F–F, and G–G). Wall reinforcing is slightly in excess of code minimum requirements. Ultimate compressive design strength used for all concrete is 2,000 psi. A plywood roof system with glued laminated beams acts as the diaphragm to distribute lateral loads to the exterior walls (fig. 3, sections A–A, B–B, and C–C). The rectangular symmetry of the building is interrupted by a reentrant corner, as shown on the roof plan (fig. 2), at the intersection of grid lines D and 6. Special lateral ties connected to struts in the diaphragm were used in two directions at this particular corner (fig. 4, detail H). An equipment platform ad-

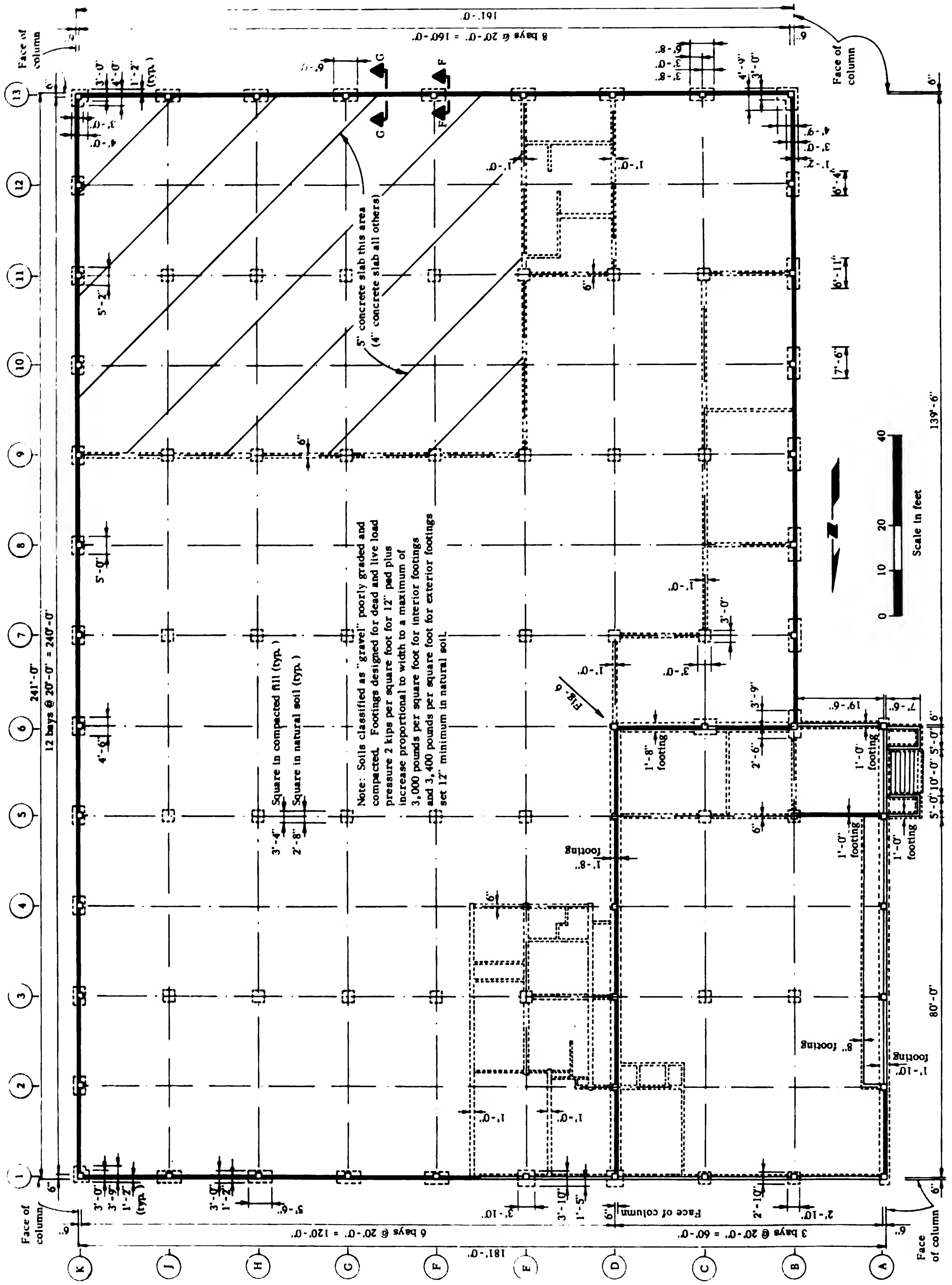

Figure 1.—Vector Electronics. Foundation plan. Precast concrete tilt-up walls; poured-in-place concrete pilasters.

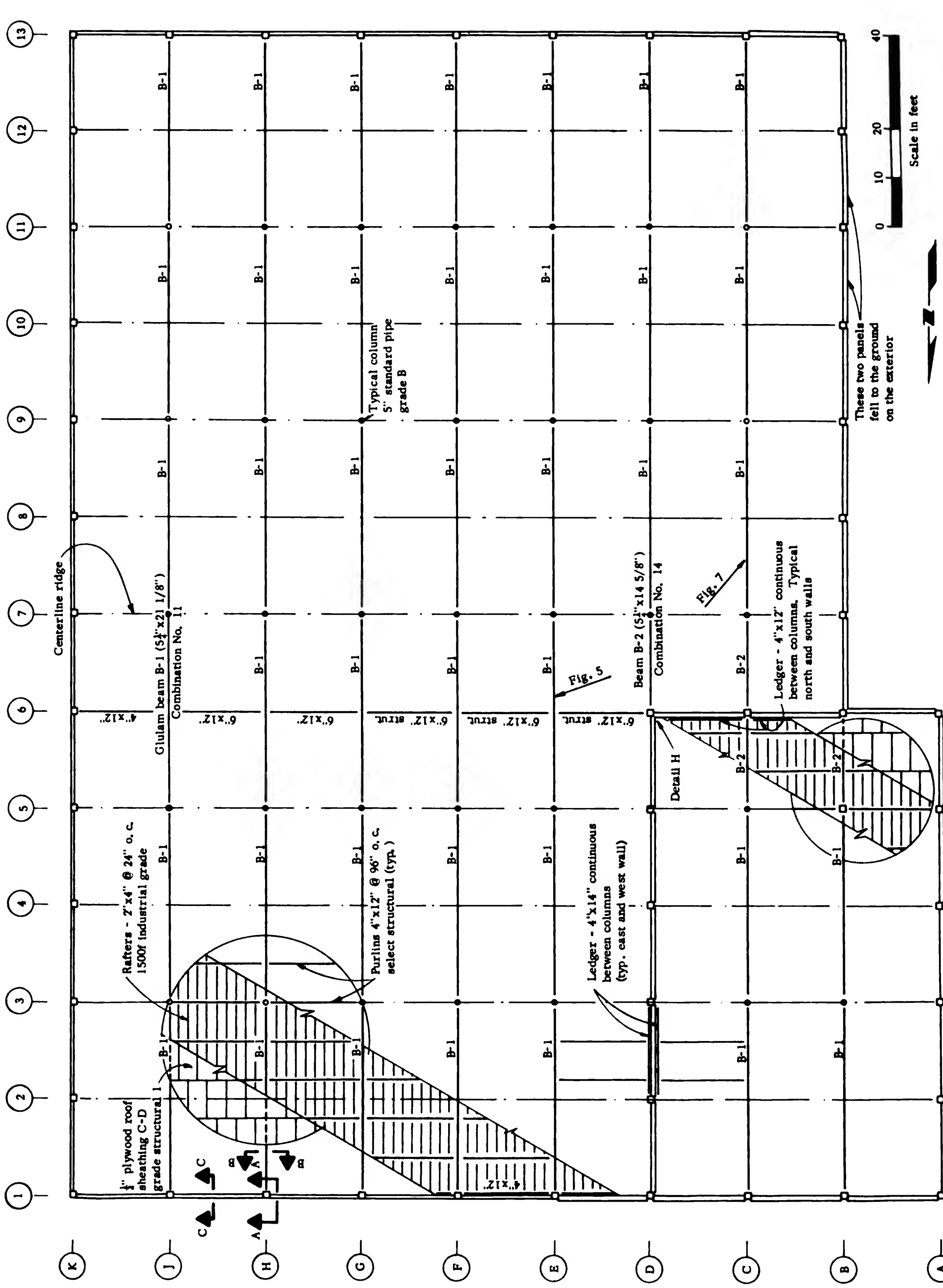

Figure 2.—Vector Electronics. Roof framing plan. Plywood roof sheathing on wood purlins and glued laminated beams.

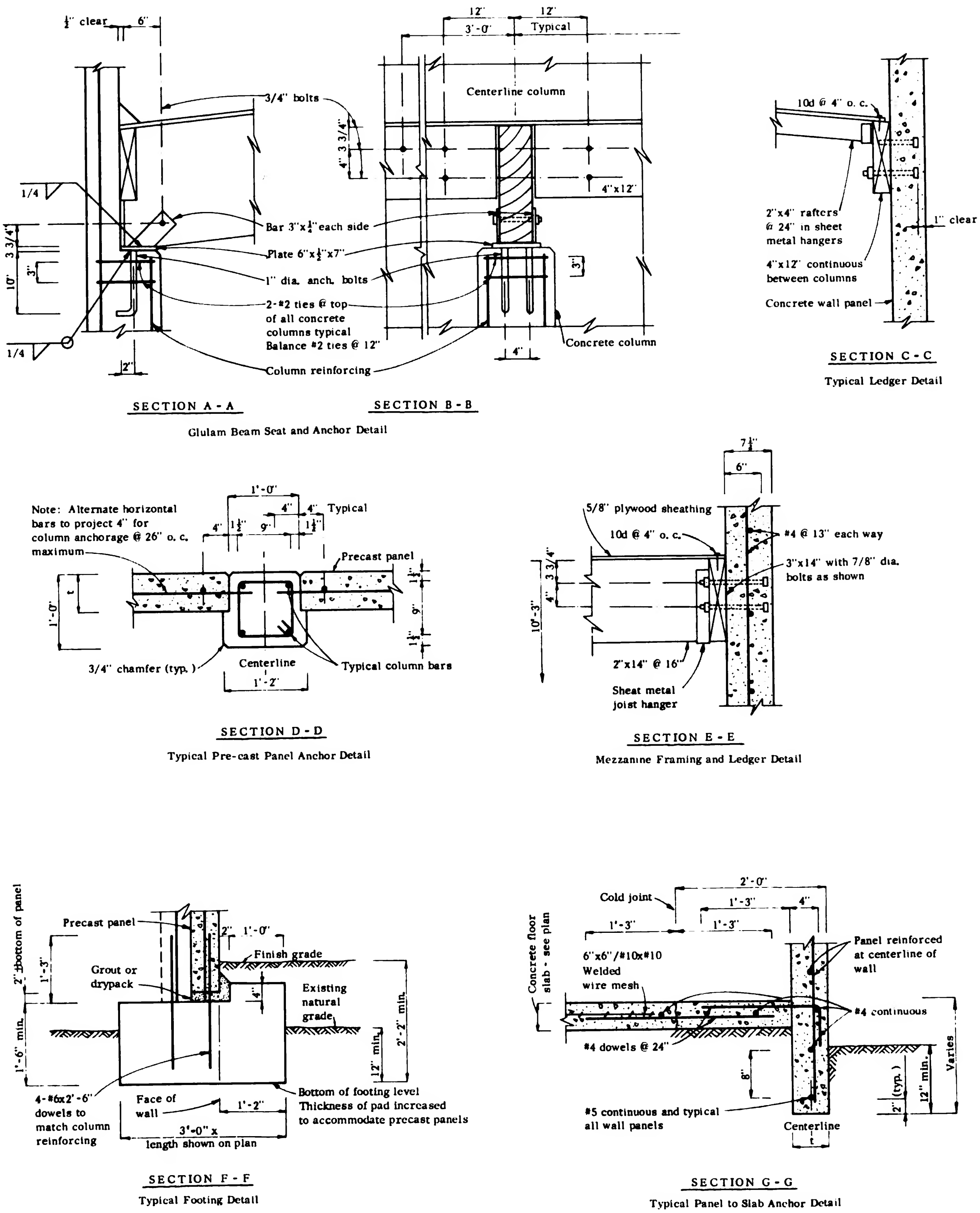

Figure 3.—Vector Electronics. Typical sections.

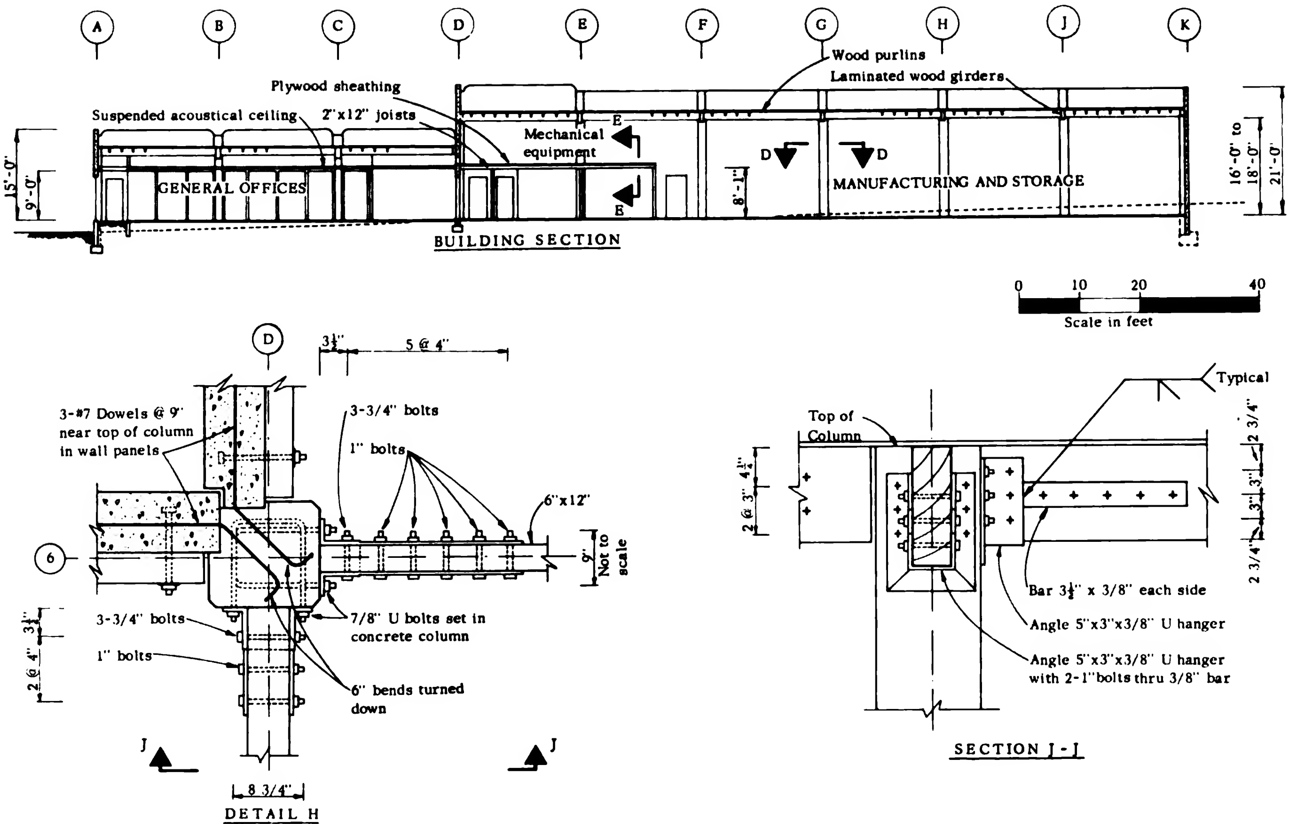

Figure 4.—Vector Electronics. Building section and details.

joins the north side of the wall on line D (fig. 2), and a storage platform adjoins the north side of the exterior wall on line B (fig. 2). These platforms were supported on the precast concrete walls (fig. 2, section E–E) and wood stud bearing partitions. An analysis of the building shows that the structure generally was designed in accordance with requirements of the Los Angeles City Building Code. Horizontal wall reinforcing steel does not appear to be spliced at columns, in accordance with code requirements.

EARTHQUAKE DAMAGE

A survey of structural damage to the building showed several types of damage and collapse. The principal cause of collapse was failure of the plywood roof diaphragm connection to the walls. Diaphragm failure consisted of nails pulling through plywood sheets or out of wood ledgers. Some ledgers split horizontally along the upper bolt line. There was some evidence of distress in the roof diaphragm in the area of the reentrant corner (fig. 5). An examination of the outside of the building showed no

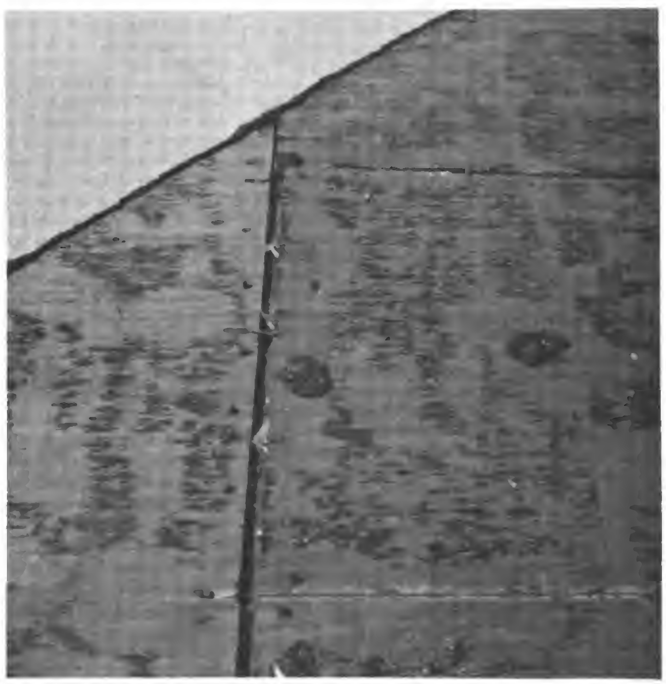

Figure 5.—Vector Electronics. Note spreading of roof sheathing over line 6 near reentrant corner. Sheets had been closely butted prior to earthquake. Wheeler & Gray photograph.

Figure 6.—Vector Electronics. Typical floor slab cracking. Wheeler & Gray photograph.

Figure 7.—Vector Electronics. Collapse of roof after connection to wall ledgers failed. Two panels fell to ground on west wall. Wheeler & Gray photograph.

evidence of vertical displacement. Interior floor slabs showed numerous east-west cracks, which varied up to an inch in width but with minor differential settlement across the line of fracture. A series of cracks, generally in an east-west direction and corresponding to the floor slab cracks, was observed in the pavement in Gladstone Avenue. Typical slab cracking is shown in figure 6. This cracking or spreading was responsible for stretching the building approximately 6 inches in a north-south direction.

The majority of wall panels on the south and east sides of the building separated from the roof diaphragm, causing the supported roof members to fall to the floor as described above (fig. 7). Wall panels at the southwest corner are shown leaning outward in figure 8. Two panels on the west side fell completely to the ground, and the balance of the dis-

Figure 8.—Vector Electronics. Wall panels leaning outward at southeast corner of building. Shoring has been installed for safety reasons. Wheeler & Gray photograph.

Figure 9.—Vector Electronics. East wall. Note horizontal crack at floorline. Floor slab has been removed and earth excavated at top of footing. Wheeler & Gray photograph.

Figure 10.—Vector Electronics. View of wall ledger at collapsed panel of west wall. Note bent perimeter nails used to connect roof sheathing and shallow embedment of expansion-type concrete anchors used. J. F. Meehan photograph.

placed panels remained upright, being restrained by the footing pads and floor slab dowels after the roof was separated. Cracking of wall panels and poured-in-place columns at about the ground line, due to vertical cantilever action, was widespread (fig. 9). Evidence of foundation failure could not be found.

Numerous glued laminated beams were pulled off their seats as column-to-beam and plywood sheathing connections yielded.

Due to cantilever action, considerable wall and column cracking occurred after failure of roof-to-wall connections. Expansion-type anchors, used in lieu of cast-in-place anchor bolts in some locations to attach wood ledgers to walls, pulled out (fig. 10). It was observed that these expansion anchors were embedded only slightly over 1 inch for $\frac{3}{4}$-inch-diameter anchors.

REPAIRS

Remedial repairs were undertaken after a consulting engineering firm, engaged for the repair work, had re-analyzed the structure. The principal repairs consisted of epoxy injection to seal wall cracks (fig. 11); strengthening diaphragm-to-wall ties by addition of strap ties bolted through the wall and connected to roof framing; welding chord reinforcing between the panels to provide for continuity; strengthening miscellaneous roof-beam connections; and adding pipe columns at certain exterior wall locations to carry vertical loads. The plywood roof sheathing adjacent to the walls was rebuilt with $\frac{5}{8}$-inch-thick sheets instead of the original $\frac{1}{2}$-inch-thick sheets.

The cost of repair was approximately 25 percent of the building value prior to damage.

CONCLUSIONS

After reviewing actual damage, it is concluded that the plywood diaphragm failed because of a combination of notable weaknesses peculiar to this building: (1) Diaphragm chord reinforcing lacked continuity required to resist chord tensile forces; (2) connections (drag ties) of roof beams into the shear walls at the reentrant corner became overstressed at splice connections along lines D and 6 (fig. 2).

RECOMMENDATIONS

1 Roof diaphragm drag ties, i.e., at the reentrant corner, should be connected entirely across buildings for calculated forces.

2 Reinforcement located at the roofline at tilt-up walls should be spliced for full continuity.

Figure 11.—Vector Electronics. Wall panels not too severely cracked were injected with epoxy as shown. Tests were made to insure penetration and effectiveness of repair. Adhesive Engineering Company photograph.

Wendell Machine Shop (7)

12685-12691 Foothill Boulevard, Sylmar

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WHEELER & GRAY
Consulting Engineers
Los Angeles, Calif.

GENERAL DESCRIPTION

This 14,500 square-foot one-story building measures 132 by 110 feet in plan and consists of a wood roof enclosed by 8-inch reinforced hollow concrete block bearing walls. It was designed in accordance with the Los Angeles City Building Code.

The plywood sheathing is supported on wood subpurlins, purlins, and glued laminated girders (fig. 1). Isolated pad footings support interior pipe column loads. Concrete block walls on the exterior are supported by continuous wall footings (fig. 1, sections A–A, B–B, and C–C). Underlying soils are compact silty sand with an assumed allowable bearing of 1,000 psf. Concrete had an ultimate compressive strength of 2,000 psi at 28 days. Lightweight concrete block units were used for the walls in which only the cells containing reinforcing were grouted.

The plywood roof was designed to act as a horizontal diaphragm to distribute seismic loads to the exterior concrete block shear walls. Typically, the roof sheathing is nailed to a 4-inch-thick ledger bolted to the walls (fig. 1, sections A–A and D–D). Horizontal steel reinforcement in the walls of the roofline provided for diaphragm chord tensile requirements.

EARTHQUAKE DAMAGE

The building was under repair at the time of the field inspection. Therefore, repair drawings and personal interviews, together with onsite observations, were used to ascertain damage.

There was severe soil movement, with vertical displacements up to 2 feet affecting approximately the northeastern one-third of the building (foundation plan, fig. 1).

The severe soil upheaval cracked the floor slab at several locations and resulted in sufficiently serious damage to the north and east walls to require their replacement (fig. 2). The northern 20 feet of the

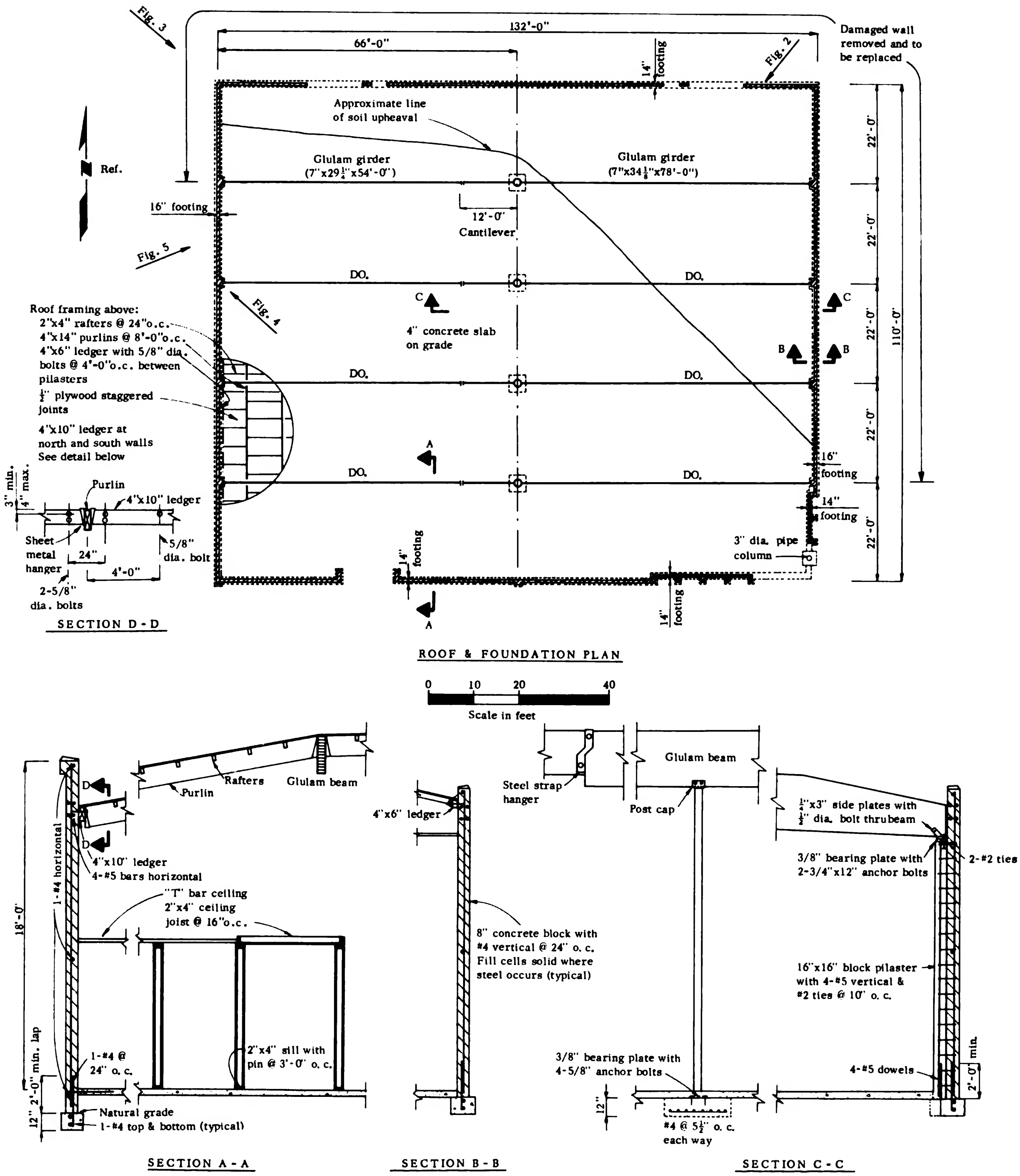

Figure 1.—Wendell Machine Shop. Roof and foundation plan.

Figure 2.—Wendell Machine Shop. North and east elevations of building. Damaged walls have been removed and will be rebuilt. Wheeler & Gray photograph.

west wall also was affected by the ground motions. Severe cracking occurred, making it necessary to replace that portion of the west wall (fig. 3). The remainder of the west wall was pushed 2 inches out of plumb, but remained in reparable condition.

The author was informed of areas where the plywood roof sheathing separated from the wood wall ledgers. It could be assumed that those areas would correspond with the areas of wall damage described in the previous paragraph.

Figure 4 shows separation of the glued laminated roof girder from the reinforced concrete block pilasters at the east and west walls. Severe spalling occurred in the upper portion of the pilasters when the forces exceeded the capacity of the connection (fig. 1, section C–C).

Figure 4.—Wendell Machine Shop. Pilaster located at west wall. Damage to upper portion at junction with glued laminated girder. Shoring of girder is still in place. Wheeler & Gray photograph.

Figure 3.—Wendell Machine Shop. Northwest corner of building. Walls to be replaced have been removed. Wheeler & Gray photograph.

Figure 5.—Wendell Machine Shop. Front (south) wall, looking northwest. This wall was not damaged although extensive window glass breakage occurred. Wheeler & Gray photograph.

The front (south) portion of the building parallel to Foothill Boulevard was not affected structurally, and the damage was limited to cracking of plaster finishes and general glass window breakage (fig. 5). The good structural performance of this portion of the building can be attributed to the stiffening action of the several wood partition walls in that area.

REPAIRS

The north and east walls and about 20 feet of the west wall were rebuilt. The remainder of the west wall was moved back into plumb and reconnected at the roof. Steel strap anchors were added along the north and south walls to provide positive attachment of 4-inch roof purlins to walls. Similarly, clip angles were added at the east and west walls at 4 feet to connect roof subpurlins to walls. All damaged concrete floor slabs were replaced or repaired.

Total repair costs to the structure approximate $42,000, which is estimated to be about 30 percent of the value of the building prior to damage.

Bennett Industries (8)

1647 Truman Street, San Fernando

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JAMES H. THOMPSON

Wilson & Thompson, Structural Engineers

Los Angeles, Calif.

GENERAL DESCRIPTION

The building site slopes about 4 feet from east to west. Foundations are spread footings designed for a maximum bearing pressure of 1,500 psf.

The building code used for design criteria was the Uniform Building Code, 1967 edition. The project was designed and built in 1970. In accordance with building code requirements, seismic design was based on a force level of 13.3 percent of the dead load of building elements, utilizing a box system. Roof diaphragm design shears were under 350 pounds per lineal foot and wall shears were about 6 psi.

The site is about 3/4 mile southwest and 1/2 mile southeast, respectively, of the surface faults and ruptures of the Sylmar and San Fernando segments of the San Fernando fault zone. The seismic force level in this area is estimated to have been 20 to 40 percent of gravity acceleration; however, ground ruptures were not noted at the building site. Building failures in the area were limited primarily to unreinforced masonry structures built prior to 1933.

The building is approximately 175 by 244 feet in size. It is one story except for a minor mezzanine area along the west wall (fig. 1). Roof construction is a typical panelized wood system with glued laminated girders, 4- by 14-inch purlins, 2- by 4-inch rafters, and 1/2-inch plywood. Wall ledgers bolted to the wall are 3 by 6 inches at the north and south walls and 3 by 10 inches at the east and west walls (fig. 1, sections A–A and B–B). Walls are constructed of 8-inch reinforced hollow concrete block units with cells filled solid with grout at 24 inches on center. One exception is the east wall which is grouted solid (all cells filled). Wall height averages about 17 feet 4 inches to the ledger bolts with a total height of about 20 feet. Floor construction is concrete slab on grade.

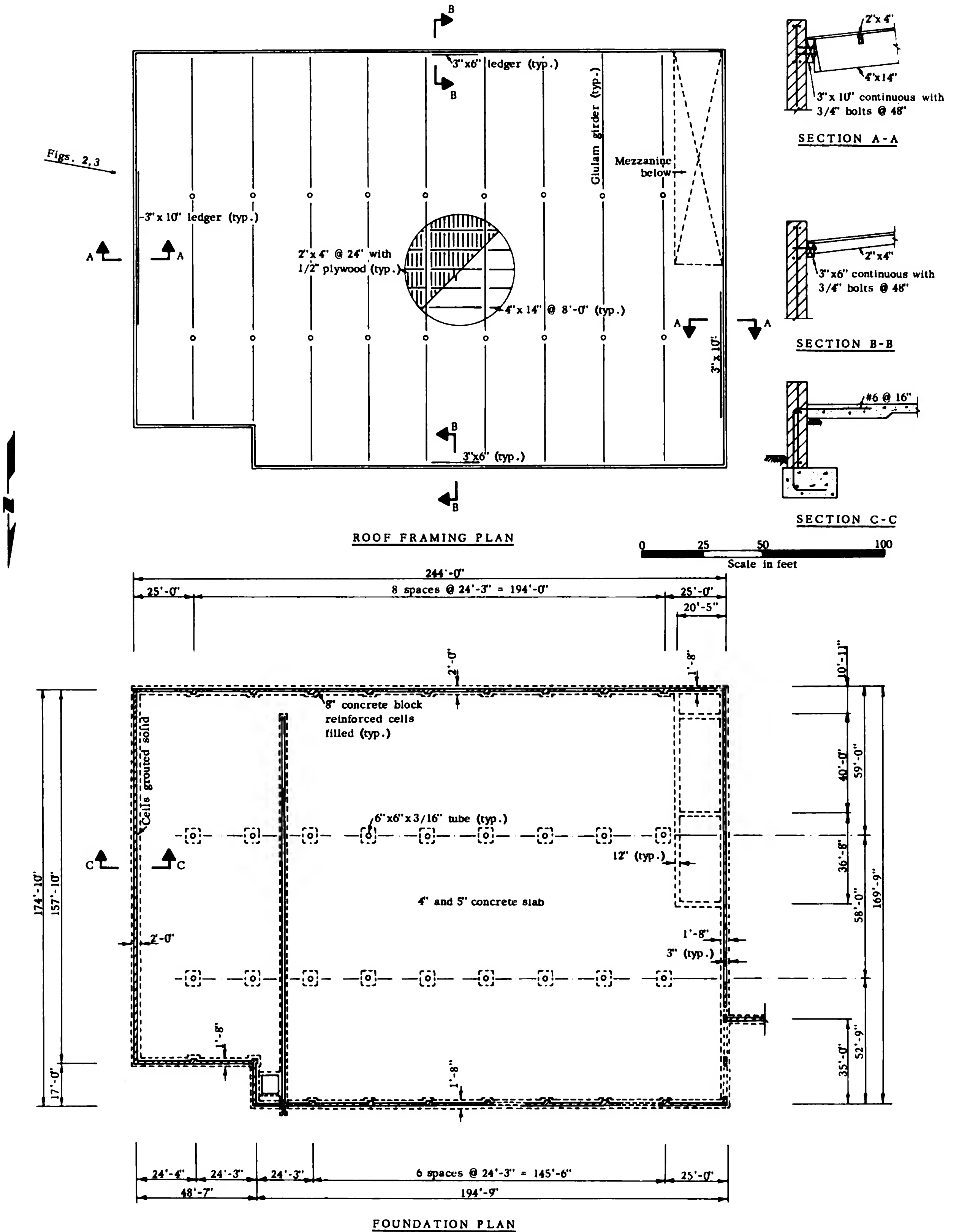

Figure 1.—Bennett Industries. Roof framing plan and foundation plan.

Figure 2.—Bennett Industries. Before repair work. Note open area at top of east wall. Rudy Gunnarson photograph.

Figure 3.—Bennett Industries, looking west, after repair work. Rudy Gunnarson photograph.

EARTHQUAKE DAMAGE

Damage was limited primarily to the east wall (figs. 2 and 3), a part of which was torn loose from the roof.

CONCLUSIONS

Discussions with building department personnel indicate the partial cause of damage at this particular point was heavy rolls of paper falling against the wall. The paper is reported to have been stacked in a storage area adjacent to the wall.

This type of hazardous condition is controlled by the State of California Department of Industrial Relations publication *General Industry Safety Orders,* issued by the Division of Industrial Safety. Section 3256. (b) states:

> Material, wherever stored, shall be piled, stacked, or racked in a manner designed to prevent it from tipping, falling, collapsing, rolling, or spreading. Racks, bins, planks, sleepers, bars, strips, blocks, sheets, shall be used where necessary to make the piles stable.

The replacement cost of the structure, prior to the earthquake, can be estimated at about $12 per square foot, or a total cost of $495,000. Repairs to the damaged wall, estimated at about $10,000, consisted of replacement of damaged material to its preearthquake condition.

RECOMMENDATIONS

It seems likely that the building could be made safer by changing the method of paper storage. A more stable type of stacking or a system of racks or retaining elements could be used to provide positive control of materials during an earthquake.

Thriftimart Market (9)

24200 Lyons Avenue, Valencia, Los Angeles County

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JAMES H. THOMPSON

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GENERAL DESCRIPTION

The building is located about 9 miles northwest of the earthquake epicenter. Foundations were spread footings, designed for a maximum bearing pressure of 1,800 psf for continuous footings and 2,000 psf for column footings. All footings were designed to bear on undisturbed natural soil. Soils were described as clayey silt, clayey sand, and lean clay.

The structure was designed and built in 1966–67. The building code used for design criteria was the Los Angeles County Building Laws, which is based on the 1964 edition of the Uniform Building Code. Seismic design was based on a force level of 13.3 percent of the dead load of building elements, utilizing a box system, and was in accordance with building code requirements.

The seismic force level in this area is estimated to have varied from 15 to 30 percent of gravity acceleration. Ground rupturing was not observed at the building site.

The building is 170 by 177 feet in size and one story high (fig. 1). There are two wood frame mezzanines at or near the south wall. Roof construction is a typical panelized wood system with tapered steel girders, 4- by 12-inch purlins, 2- by 4-inch rafters, and ½-inch plywood, Structural I Grade with exterior glue. Plywood is nailed with 10d nails at 12 inches on center at interiors of sheets, at 3 inches on center at edges of sheets, and at 2 inches on center at boundaries of the diaphragm (along exterior wall lines). Wall construction is reinforced brick—11 inches thick at the south wall and reinforced with No. 4 vertical bars at 20-inch spacing and No. 4 horizontal bars at 16-inch spacing; 8½ inches thick at the west and north walls and reinforced with No. 4 bars at 24-inch spacing in each direction; 12 inches thick at the east wall and reinforced with No. 4 vertical bars at 12-inch spacing and No. 4 horizontal bars at 24-inch spacing. Floor construction is concrete

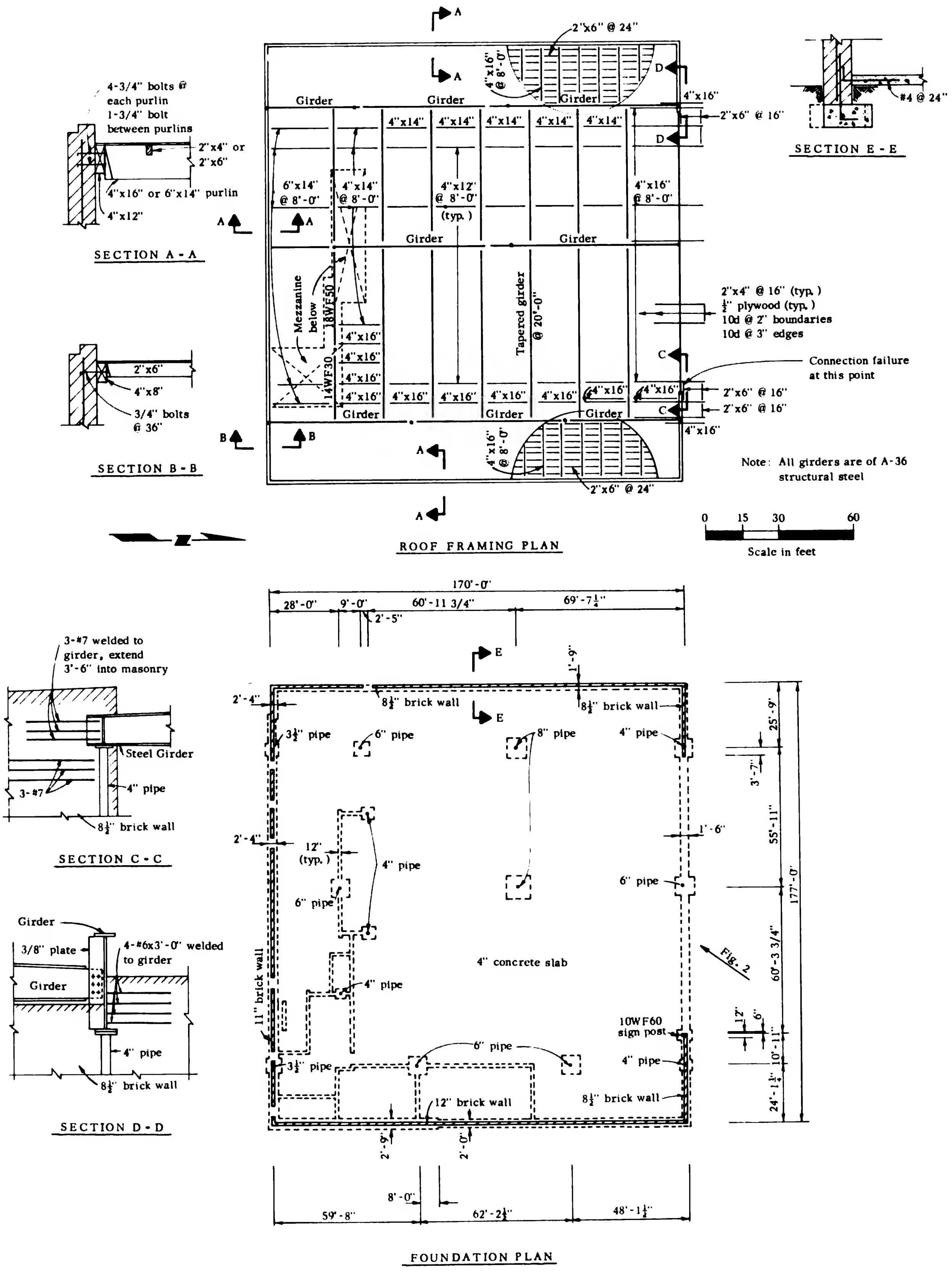

Figure 1.—Thriftimart Market. Roof framing plan and foundation plan.

slab on grade. Walls are anchored to the floor slab with No. 4 dowels at 24-inch spacing.

EARTHQUAKE DAMAGE

The north wall has a masonry section at each corner, about 35 feet long at the east end and about 29 feet long at the west end. These wall segments, along with the south wall, resist east-west seismic forces and are tied together by tapered steel girders that act as chord and collector members for the roof diaphragm (figs. 1 and 2). One of the girders is connected to the east wall segment by three No. 7 reinforcing bars that are welded to the web of the girder and extend (horizontally) about 3 feet 6 inches into the masonry wall (fig. 1, section C–C). The bars lap with three No. 7 bars, placed at a lower elevation, that extend about 14 feet along the wall. This connection pulled loose, spalling the brick away from the length of the three No. 7 welded dowels, and vertically along the edge of the wall where a pipe column is embedded to support the vertical load of the girder (fig. 3). The failure appears to have been caused by a combination of bond and shear overload at the connection. Since the three continuous bars were placed below the welded dowel bars, the total load was transferred by shear through a net section of the wall.

Figure 2.—Thriftimart Market. Front (north) wall. Note masonry wall segments at each corner. J. Kesler photograph.

Figure 3.—Thriftimart Market. Strut connection to brick shear wall. Exposed are three No. 7 bars about 14 feet long. J. Kesler photograph.

Using the code seismic force level, the force at the connection, assuming half the tributary load went to each of the two wall segments, would be about 18,000 pounds; shear in the wall, assuming 3 feet 6 inches effective length, would be about 50 psi. The bond stress on the three bars would be 53 psi. These figures are approximate only because of the unknowns involved with relative stiffnesses of walls, struts, connections, direction of forces, and other factors that would affect distribution. From this analysis, it is evident that a connection failure should not occur until actual force levels reached several times the code force level. The damage was repaired by raising the three long bars and welding them directly to the three dowels. A continuous stress path thus was obtained, providing uniform force distribution over a longer length of wall.

The tapered girder is connected to the west wall segment with four No. 6 dowels, with a 3-foot (horizontal) embedment into the wall (fig. 1, section D–D). There was no significant structural distress at this connection. The connection is slightly more flexible than the one at the opposite (east) end, because the bars are not welded directly to the strut girder but to another girder, which is at right angles to the strut. It, in turn, supports the strut girder with a bolted connection. Some bending within the structural steel connection would be required to transfer loads with a corresponding flexibility. A substantial part of the total force may have been distributed to the other end owing to the differential connection flexibilities. This connection was opened partially to check for signs of distress; nothing significant was observed.

Along the south wall, the connection between the plywood diaphragm and the 4- by 12-inch wall ledger suffered some distress. The ledger partially

split because of the separation force between the plywood and the ledger, perpendicular to the wall. The ledger was bolted to the wall with four ¾-inch anchor bolts at each purlin (8 feet on center), and with one additional ¾-inch bolt between purlins (fig. 1, section A–A). The top row of bolts was placed 4 inches down from the top of the ledger. The ledger failed in cross-grain bending (tension) as the plywood pulled on the top of the member. The split ledger was replaced. Damage to the wall was inconsequential.

CONCLUSIONS

The failure of the east wall-girder connection is fairly typical and falls in the category of building continuity. Where force-resisting elements of different rigidities and masses are connected by struts or continuity ties, the rational assumptions of seismic design, assuming static loads, lose much of their reliability. Dynamic conditions of load and element response can be vastly different from the static condition assumed in the analysis.

Distress and damage at the plywood diaphragm connection to the wood ledger were quite common for buildings of this type. In some cases the plywood pulled off the ledger with a nail bearing-shear failure. In this case the plywood held and the ledger split.

The replacement cost of the building, prior to the earthquake, can be estimated at about $18 per square foot, or a total of $540,000. Structural repair costs have been estimated at about $15,000.

W. T. Grants (10)

19419 Soledad Canyon Road, Saugus
Los Angeles County

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JAMES H. THOMPSON
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GENERAL DESCRIPTION

This building is in the Saugus-Newhall area, about 7 miles northwest of the earthquake epicenter. The site slopes slightly, about 3 feet, from east to west. Native soil is medium to coarse gravelly sands. Foundations are spread footings designed for a maximum bearing pressure of 4,000 psf.

The project was designed and built in 1969–70. The building code used for design criteria was the Los Angeles County Building Laws, 1968 edition, which is based on the Uniform Building Code, 1967 edition. In accordance with building code requirements, seismic design was based on a force level of 13.3 percent of the dead load of building elements, utilizing a box system. Roof diaphragm shears were about 425 pounds per lineal foot, and wall shears were approximately 9 psi at code force levels.

The one-story building is approximately 188 by 245 feet in size with added structures on the west side of about 50 by 109 feet (fig. 1). There is a 60- by 65-foot wood frame mezzanine along the east wall; another mezzanine is about 30 by 30 feet in size. Roof construction is a typical panelized wood system with glued laminated girders, 4- by 16-inch purlins, 2- by 4-inch rafters, and ½-inch plywood, Structural I Grade with exterior glue. Plywood is nailed with 10d nails at 12 inches on center at interiors of sheets, at 6 inches on center at edges of sheets, and at 4 inches on center at boundaries of the diaphragm (along exterior wall lines). The plywood roof sheathing is nailed to wood ledgers that are bolted to the walls (fig. 1, sections A–A and B–B). Walls are constructed of 10-inch reinforced hollow concrete block with only the reinforced cells grouted. Vertical reinforcing is No. 4 bars at 24 inches on center, and horizontal reinforcing is No. 4 bars at 48 inches on center with extra reinforcing added along the top of the wall at the roofline. Wall heights are about 22 feet. Floor construction is concrete slab on

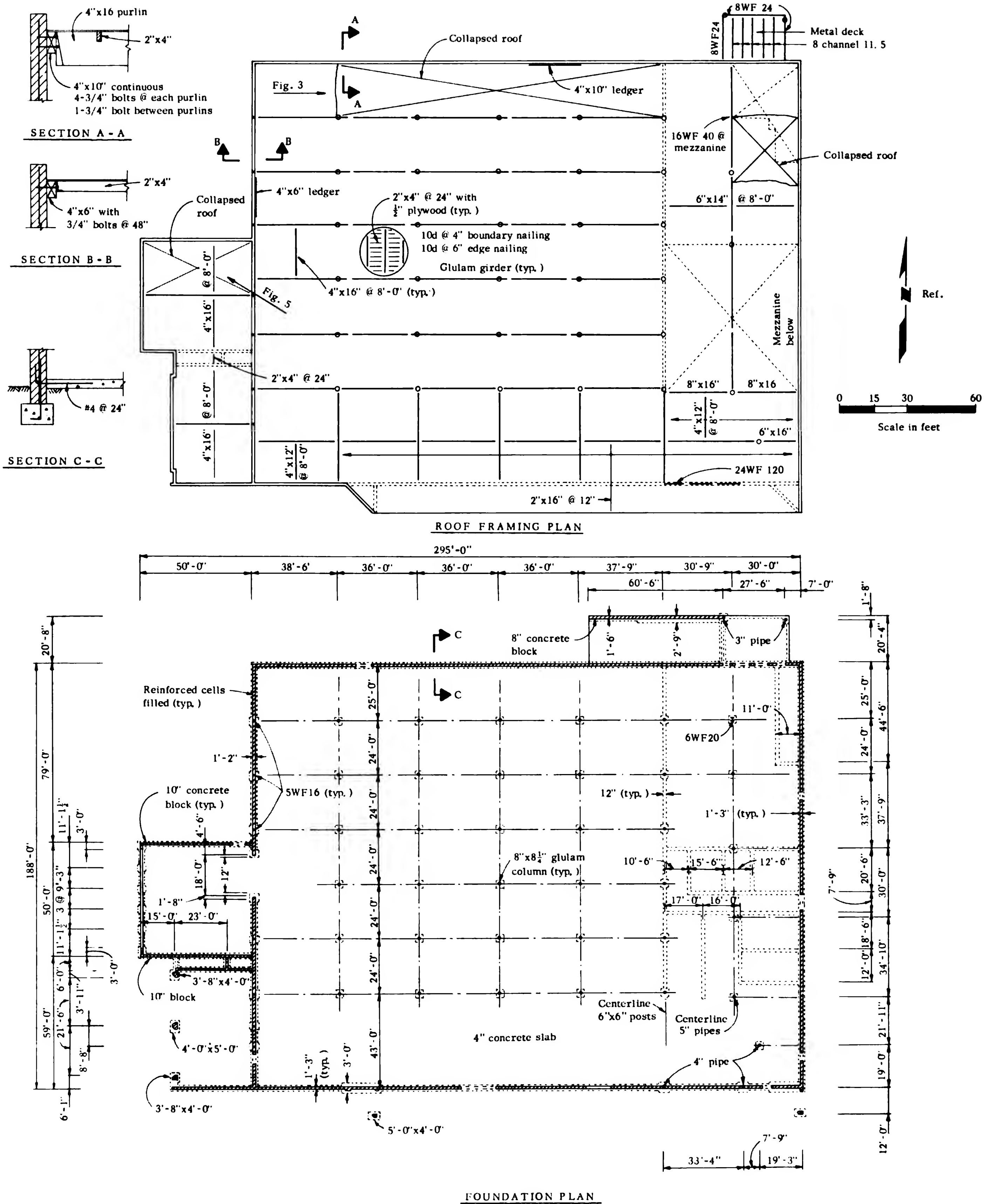

Figure 1.—W. T. Grants. Roof framing plan and foundation plan.

grade. Walls are anchored to the floor slab with No. 4 dowels at 24 inches on center.

EARTHQUAKE DAMAGE

Structural damage was located primarily in the walls where they are connected to the plywood roof system through the ledger. Failure of the wall-to-roof framing connection allowed the wall to separate from the roof construction. This movement pulled the joist hanger supports away from the ends of the framing members and they fell, pulling loose from their support at the other end at the same time (figs. 2 and 3). This type of failure usually was triggered by one or more of the following types of rupture:

1 The plywood pulled loose from the ledger, with a combination shear-bearing failure at the nail, with the nail force acting toward the edge of the plywood. Usually the plywood nail edge distance is from 3/8 to 3/4 inch.

2 The plywood nailing remained intact, but the force exerted on the top of the ledger split the ledger in cross-grain bending (tension).

3 A combination of the first two modes of failure occurred. Excessive ledger rotation or displacement contributed to the nail forces, causing the nail to fail.

CONCLUSIONS

An investigation into the pattern of failures and the type of plywood nailing suggests another possible

Figure 2.—W. T. Grants (foreground). Note three areas (dark) of roof collapse. Builder's Emporium (11) is just above W. T. Grants. Department of County Engineer photograph.

Figure 3.—W. T. Grants. Collapsed roof along north wall. J. F. Meehan photograph.

mode of failure. Generally, the areas of fallen roof are where the 4- by 16-inch purlins span between the exterior wall and the girder on the first column line inward from the wall. Typically, there is no framing continuity through the girder line, except for the plywood that is staggered over the girder. A line of separation (diaphragm tension failure) occurred at and parallel to the girder. With edge nailing at 6 inches on center and boundary nailing at 4 inches on center, the number of nails required to fail at the girder line is approximately 20 for each tributary width (8 feet) between the purlins. The number of nails required to fail along the wall line is 24. Since alternate boards are staggered over the girder, eight of the 20 nails at that point are required to be installed in one-half the width of a 2 by 4, or about 3/4 inch. Along the wall, the boundary nails are in a 4X member and the full width is available for nailing. The reduced edge distance (about 3/8 in.) and reduced member width may reduce substantially the nail strength. It is suggested that the lateral load supporting system, provided by the roof framing for the top of the wall for forces perpendicular to the wall, may actually have begun to fail at the first girder inward from the wall line. The fewer number of nails and, in some cases, reduced edge distance, may be the weak link in the stress line. As the plywood begins to separate along the girder line, the purlins are pulled off their hanger seats and drop. The added force of the falling roof elements pulled the plywood-ledger connection apart at the wall line, and the whole roof area collapsed.

In either case (wall line first or girder line first), the roof failed and damaged the wall. This occurred

Figure 4.—W. T. Grants. Collapsed roof at west wing. J. Kesler photograph.

Figure 5.—W. T. Grants. Collapsed roof at west wing. J. F. Meehan photograph.

along the north wall for a length of about 145 feet, along the north wall of the west wing for about 50 feet, and along the east wall for a length of about 30 feet (figs. 2, 3, 4, and 5). In each of these areas the 4- by 16-inch or 6- by 16-inch purlins were framed perpendicular to the wall. The wall was forced out of plumb by as much as 8 to 9 inches. Several other areas of the roof diaphragm sustained cracked roofing and distorted plywood. An interior wood stud bearing wall supporting the mezzanine floor joists displaced and required repairs.

The seismic force level in this area was estimated to vary from 20 to 40 percent of gravity acceleration. Ground rupturing was not observed. An adjacent building (Builder's Emporium) suffered significant structural damage; however, other stores in the same shopping center did not sustain significant damage. In a nearby shopping center on the opposite side of the street, heavy damage was observed where a ground surface rupture occurred adjacent to, and through, several buildings.

REPAIRS

The concrete block walls were repaired by jacking them back into place in some areas, and by replacing them with new materials in other areas. Additional ledger bolts through the walls, with steel plate washers both inside and outside, were installed. Bolted continuity ties through the first girder line parallel to the walls were installed as part of the repair work.

The replacement cost of the building, prior to the earthquake, can be estimated at about $16 a square foot, or a total of $875,000. Repair costs are estimated at about $90,000, of which about $40,000 was for ceiling and other architectural repairs.

Builder's Emporium (11)

19407 Soledad Canyon Road, Saugus
Los Angeles County

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JAMES H. THOMPSON
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GENERAL DESCRIPTION

This building is located in the Saugus-Newhall area, about 7 miles northwest of the earthquake epicenter. The site slopes slightly from east to west. Native soil is medium to coarse gravelly sands. Foundations are spread footings on natural soil and on fill. Maximum design bearing pressure is 4,000 psf for natural soil and 3,000 psf for fill.

The project was designed and built in 1968–69. The building code used for design criteria was the Los Angeles County Building Laws, 1968 edition, which is based on the Uniform Building Code, 1967 edition. In accordance with building code requirements, seismic design was based on a force level of 13.3 percent of the dead load of building elements, utilizing a box system.

The one-story building is approximately 170 by 185 feet with some added appendages along the north wall (figs. 1, 2, and 3). There is a small wood frame mezzanine near the north wall and a raised portion of roof at the west wall. Roof construction is a typical panelized wood system with glued laminated girders, 4- by 14-inch purlins, 2- by 4-inch rafters, and ½-inch plywood. Plywood is nailed to supports with 8d nails at 12 inches on center at interiors of sheets, at 6 inches on center at sheet edges, and at 4 inches on center at boundaries of the diaphragm (along wall lines). The boundary nailing along the south wall is 8d at 2½ inches on center. Walls are constructed of 10-inch reinforced brick. Reinforcing is No. 4 bars at 20-inch spacing each way (horizontal and vertical). Wall heights are about 22 feet. Floor construction is concrete slab on grade. Walls are anchored to the floor slab with No. 4 dowels at 20-inch spacing.

EARTHQUAKE DAMAGE

Structural damage was confined primarily to roof framing and the wall-roof interface connection. The

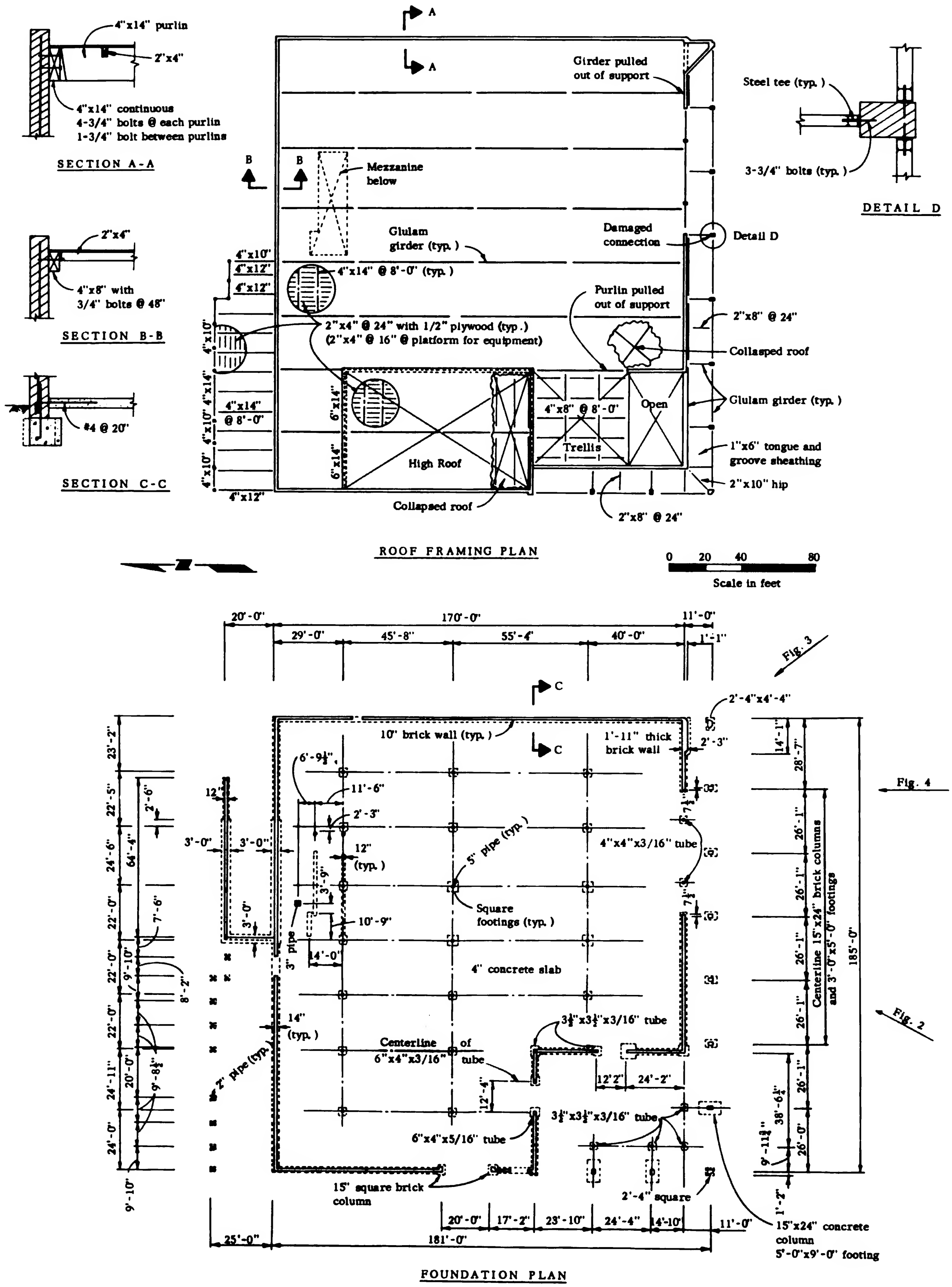

Figure 1.—Builder's Emporium. Roof framing and foundation plan.

Figure 2.—Builder's Emporium, looking north at front. J. Kesler photograph.

4- by 14-inch ledger along the east wall was split for a length of about 140 feet. The ledger was bolted to the wall with four ¾-inch bolts at each 4- by 14-inch purlin (at 8 ft on center) with one additional bolt between purlins (fig. 1, section A–A). The top row of bolts is placed 3½ inches down from the top of the ledger. The force generated by the weight of the brick wall itself, and its acceleration normal to the plane of the wall, tended to separate the top of the wall from roof attachments. This force split the wall ledger in cross-grain bending as it was transmitted from the wall to anchor bolts, from the bolts to ledger, and from the ledger to plywood sheathing (fig. 1, section A–A). The same condition occurred along the west wall, at the high roof, for a length of about 16 feet. A portion of the high roof collapsed (sagged) in its interior where a girder pulled loose from its support on a steel column. Movements of members that were large enough to dislodge them from their bearing supports were apparently caused by diaphragm flexibility resulting from different levels of roof construction and the corresponding diaphragm discontinuity. Several other members, girders, and purlins separated from their supports and caused local roof failures.

Figure 3.—Builder's Emporium. South wall. Note shores adjacent to columns. J. Kesler photograph.

Girders (7 by 18 in.) along the south wall canopy line were loosened from their supports at the masonry columns at three locations (fig. 1, section D–D, and fig. 4). The seismic axial forces in the girders were large enough to fracture the masonry column by pullout (tension) on the anchor bolts and by compression against the column face. The masonry was fractured and loosened to the extent that spalling occurred at the face of the column.

Figure 4.—Builder's Emporium. South wall girder connection with outside masonry spalled off. J. Kesler photograph.

REPAIRS

The distressed portions of the roof structure were replaced with reinforced connections. The split ledgers were removed and replaced with bolts through the wall—plate washers on the outside wall face and at the inside ledger.

Horizontal seismic force levels in this area were estimated to be from 20 to 40 percent of gravity acceleration. Ground rupturing was not observed at this site; however, surface cracking and lateral differential displacements were noted at a nearby shopping center on the opposite side of the street. Significant

structural damage to buildings occurred at that location.

W. T. Grants store, adjacent to Builder's Emporium, suffered similar damage to walls and roof-wall connections. Other stores in the shopping center did not have significant structural damage.

The replacement cost of the building, prior to the earthquake, can be estimated at about $16 per square foot, or a total of $560,000. Repair costs are estimated at about $25,000.

CONCLUSIONS

The lack of continuity in roof diaphragms, where a portion of the roof is raised to a higher elevation, is a problem in earthquake-resistant construction. Excessive diaphragm (and connected members) deflections in the plane of the diaphragm around the area of discontinuity have occurred. The upper-level diaphragm may be unusually flexible since it does not have a connected shear wall extending to grade.

Alpha Beta Market (12)

13570 Eldridge Avenue, Sylmar

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GENERAL DESCRIPTION

This building is located near the toe of the San Gabriel Mountains, on gently sloping ground about 1 mile south of the Veterans Administration Hospital. It was designed to comply with the requirements of the Los Angeles City Building Code and was built in 1961.

The general nature of the soil in this area is alluvial. A soils investigation was conducted and the borings performed revealed that the soils on the site generally are composed of sand-silt-gravel mixtures. The bearing value used for the design of the foundation was 1,500 psf at a depth of 2 feet with an increase of 200 psf for each additional foot of depth.

This building is one story in height with a small mezzanine along the north side. East and west of the market are lower roof areas occupied by several different shops. They are attached to the main building and can be considered as extensions to the main structure (figs. 1 and 2).

The roofs consist of plywood supported by wood rafters and purlins, which in turn are supported by steel beams and tapered steel carrying girders, which bear on columns of three types: steel pipes, steel-rolled sections, and reinforced concrete (fig. 2). Quality of the plywood used is not known. However, exterior-type glue was specified.

Six-inch precast concrete wall panels approximately 22 feet in height enclose the market on the east, west, and north sides (fig. 4, section 16–16; fig. 5, section 18–18). The south wall is glass above a brick apron wall (fig. 4, sections 12–12 and 13–13). Two 8½-inch reinforced brick shear walls are set back approximately 17 feet from the glass line (fig. 1 and fig. 3, line B). The other buildings are enclosed by a combination of wood stud walls with a stucco finish and glass (fig. 5, section 17–17; fig. 6, section 23–23).

Foundations are reinforced concrete continuous

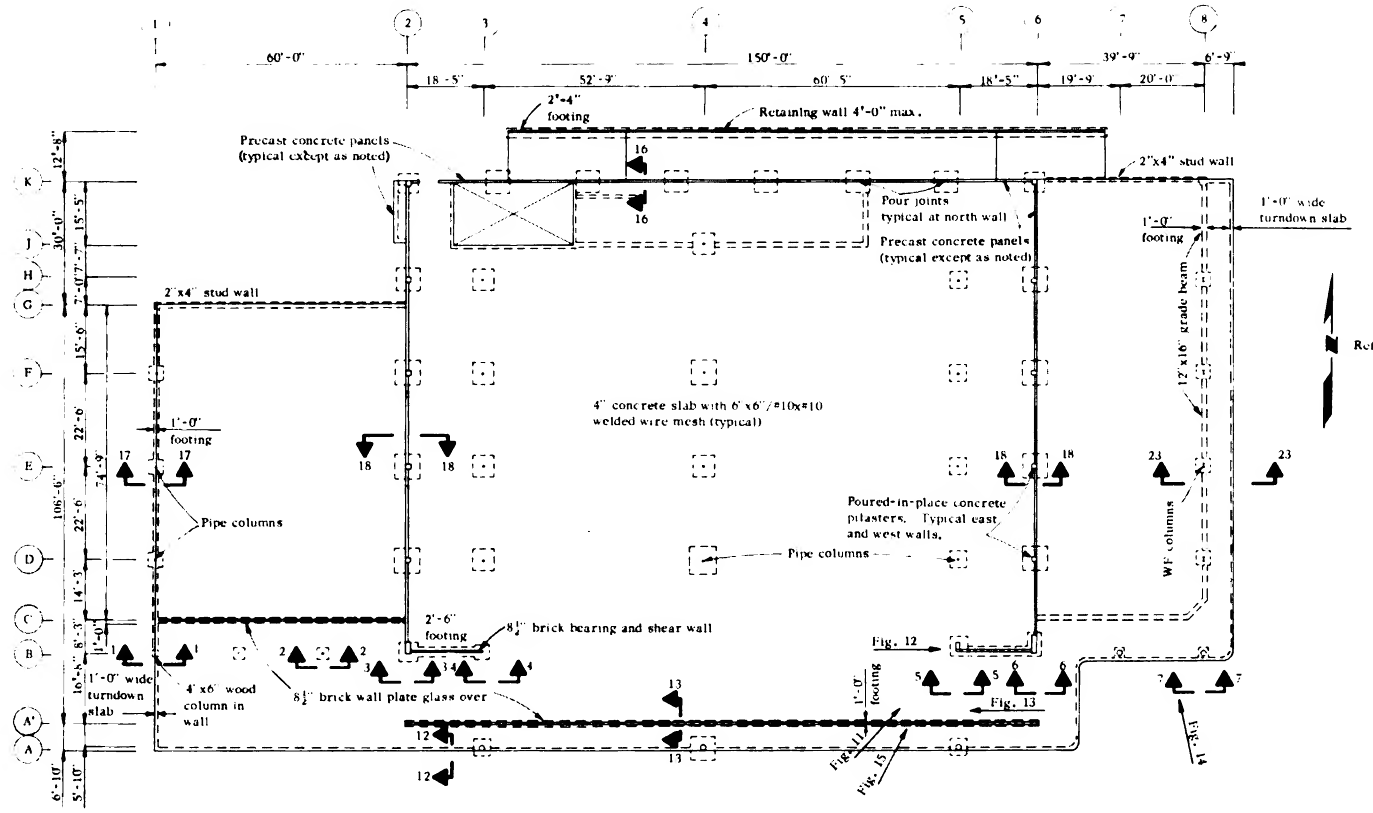

Figure 1.—Alpha Beta Market. Foundation plan.

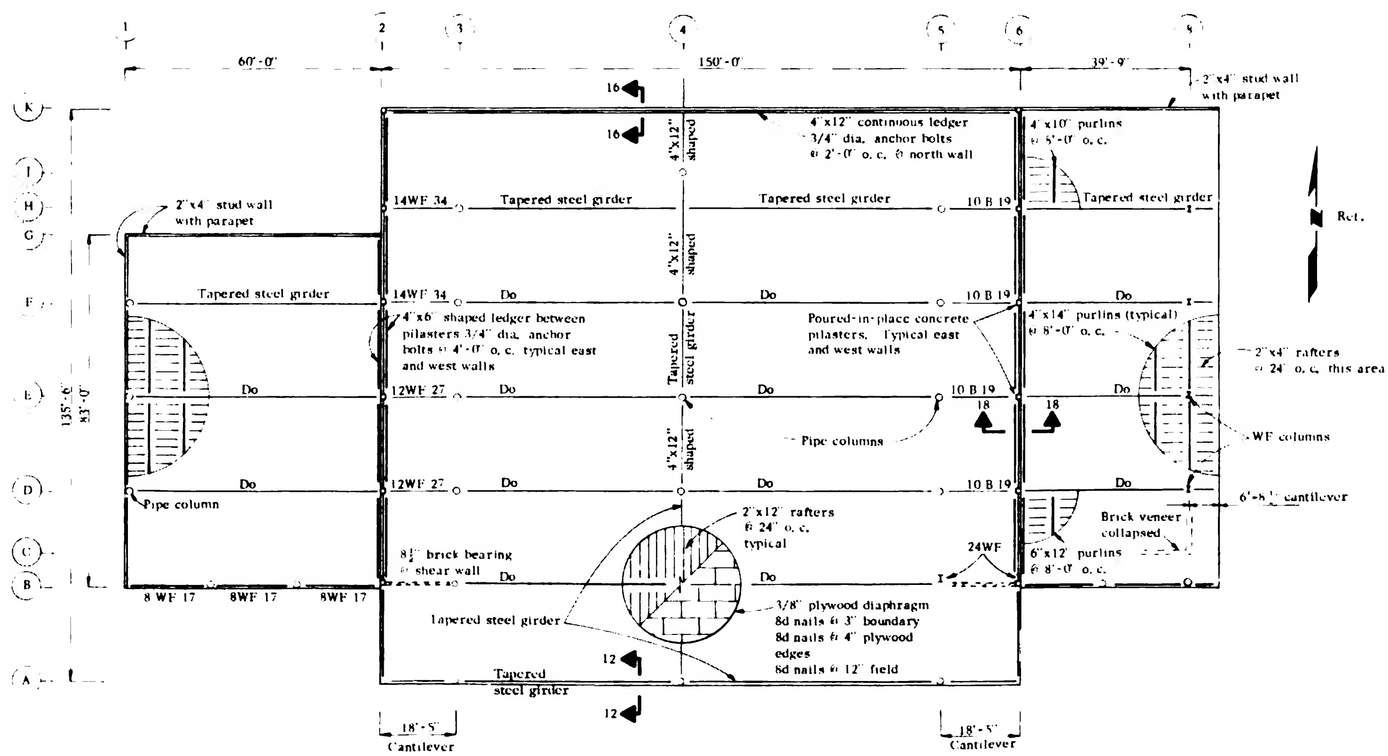

Figure 2.—Alpha Beta Market. Roof framing plan.

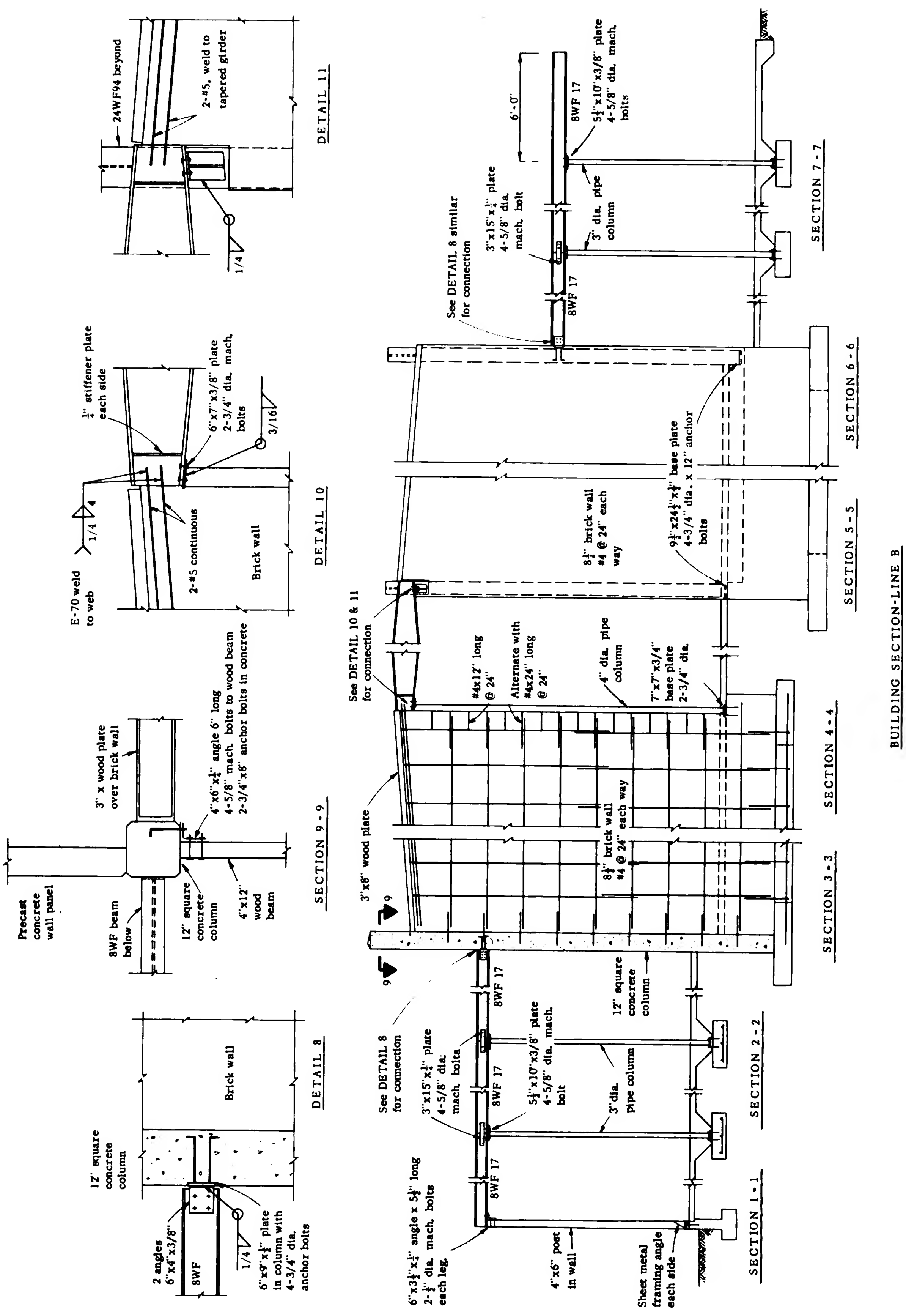

Figure 3.—Alpha Beta Market. Building section, line B.

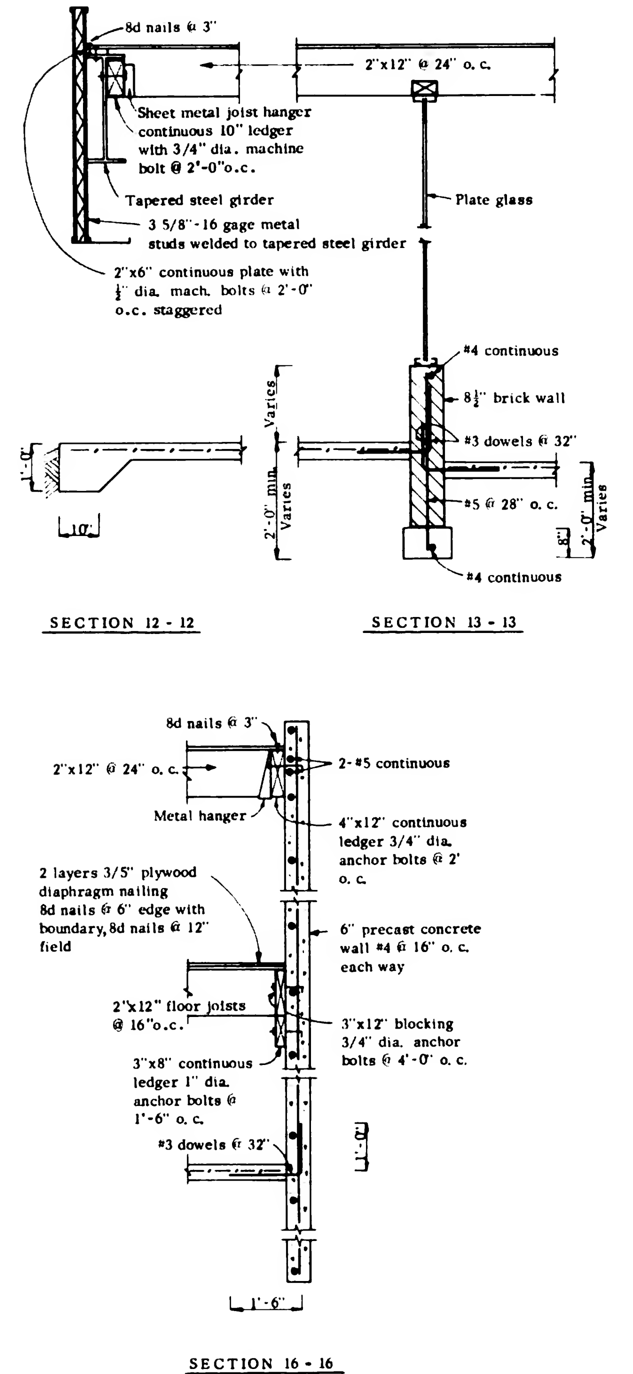

Figure 4.—Alpha Beta Market. Sections 12–12, 13–13, and 16–16.

footings under stud and brick walls, and spread footings under isolated columns (fig. 1).

The specified ultimate compressive strength of concrete used in this structure was 2,000 psi at 28 days.

For lateral forces, the ⅜-inch plywood roof was used as a horizontal diaphragm. A 4-inch-thick wood ledger bolted to the walls and nailed to the roof plywood with 8d nails at 3 inches on center provides the connection between the concrete walls and the diaphragm (fig. 4, section 16–16). Seismic forces are delivered to the two brick shear walls on line B of the main building through a tapered steel girder and wide-flange steel beams working as ties or drag struts. Steel bars and anchor bolts project out of walls and are connected to the steel drag struts by means of welding and bolts for a positive transfer of forces (fig. 3, details 10 and 11).

The building extension at the east side of the market has steel wide-flange columns supporting the roof along the east side (fig. 6, section 23–23). The

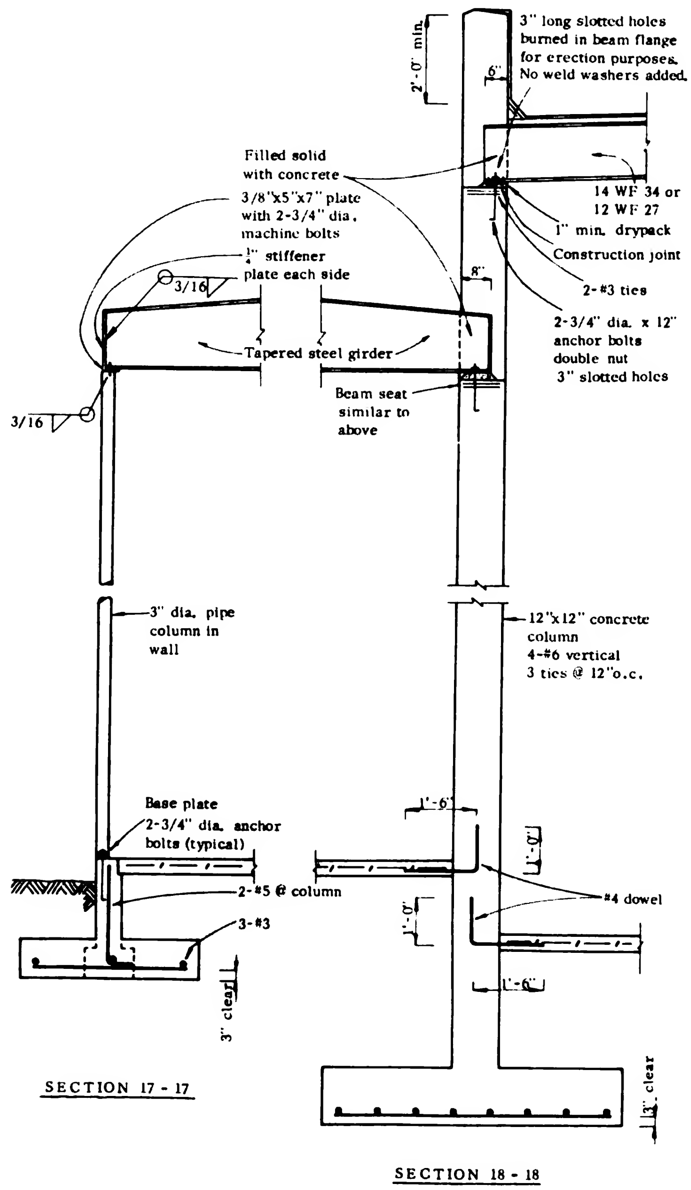

Figure 5.—Alpha Beta Market. Sections 17–17 and 18–18.

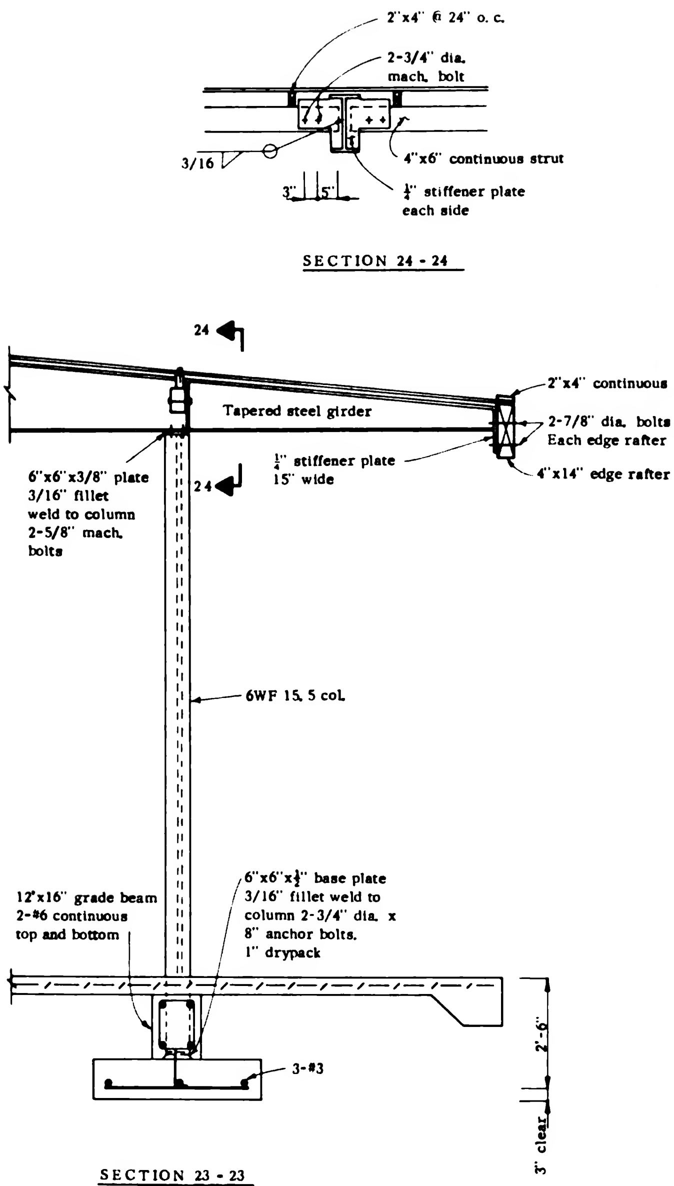

Figure 6.—Alpha Beta Market. Sections 23–23 and 24–24.

columns were designed to cantilever from a concrete grade beam foundation to resist tributary horizontal roof forces.

The building extension on the west side has solid wood stud and plaster walls on two sides and the concrete wall of the main building on line 2 (fig. 1; fig. 5, section 17–17). Lateral forces are resisted by these walls and by the 9½-inch brick shear wall of the main building (fig. 1, line B).

Overall lateral force design appears to comply with code requirements.

EARTHQUAKE DAMAGE

The ground disturbance in this area was minor.

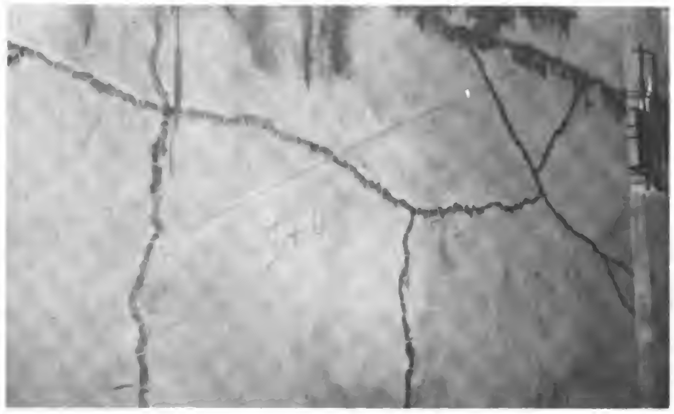

Figure 7.—Alpha Beta Market. Epoxy-patched cracks in precast concrete wall panel at line 6. Wheeler & Gray photograph.

The floor slab exhibited a ¼-inch-wide crack the full width of the building in an east-west direction. Vertical ground displacement was not evident.

At the time of the field trip, work to repair the earthquake damage was in progress; some phases of this work already were completed, while others still were under way.

Reinforced precast concrete panel walls were damaged extensively by cracking of the concrete. Even though the majority of the cracks were already repaired, they still were clearly visible (fig. 7). A definite pattern of cracking could not be detected, as the cracks occurred in vertical, horizontal, and diagonal directions. Some separation between the poured-in-place pilaster and the precast panels could be seen, but it could not be established if they were earthquake-induced or existed previously.

The poured-in-place concrete pilasters (wall columns), located between the precast wall panels, almost without exception suffered heavy damage at the points where the steel beams from the higher roof and the tapered steel girders from the lower roof connect to them (fig. 5, section 18–18). It is known that the girder anchor bolts were connected through 3-inch-long slotted holes, burned in the bottom flanges of the girders for erection purposes. Considerable movement between girders and wall pilasters undoubtedly contributed to the damage at these connections. Serious cracking and spalling of the concrete, which in some instances extended to adjacent panels, took place at those points (figs. 8, 9, and 10).

The two reinforced brick walls on the south side of the main building, providing the lateral stability in an east-west direction, sustained severe cracking

Figure 8.—Alpha Beta Market. End of tapered steel girder from adjacent lower roof. Wheeler & Gray photograph.

Figure 9.—Alpha Beta Market. Damage inflicted to wall and pilaster by end of tapered steel girder. Temporary shoring at left. Wheeler & Gray photograph.

and partial breakage of bricks (figs. 11 and 12). The author was not able to see the walls since they had already been removed, but personal descriptions indicated that the major extent of the damage took place in the lower portions of the wall near the floor slab.

The north precast concrete wall separated from the roof diaphragm and moved out about 6 inches, damaging the ends of the roof rafters.

In determining the mode of wall-to-roof diaphragm failure, along line K, it would appear that two occurrences took place and, perhaps, simultaneously: (1) Nails attaching the plywood to the wood ledgers bolted to the walls were pulled through the edge of the sheathing, and (2) some wood ledgers split at the bolt line. It is of interest to note that, in this building, roof collapse did not follow the separation of the wall from the roof. The north wall ledger, although damaged, remained sufficiently connected to the wall to keep the roof from collapsing.

An exterior section of brick veneer collapsed at the southeast corner of the lower building located east of the market (fig. 13).

Widespread damage to nonstructural items occurred throughout the building. Most of the glass window wall at the front of the market cracked and collapsed (fig. 14). Acoustical tile soffits buckled and spalled off, wall finishes cracked, lighting fixtures broke loose from their supports and fell to the floor, and most of the market merchandise spilled from the shelves.

From known damage, it is estimated that horizontal accelerations were in the 40 percent of gravity range.

Figure 10.—Alpha Beta Market. Top of concrete pilaster at connection with steel beam. Wall line 6. Wheeler & Gray photograph.

REPAIRS

As mentioned previously, corrective work and repairs were under way at the time of this writing. The structure was being restored to its preearthquake condition with a few minor modifications that are expected to improve the lateral stability of the building.

The reinforced brick shear walls were removed completely and replaced by reinforced gunited con-

Figure 11.—Alpha Beta Market. Front (south) shear wall. D. F. Moran photograph.

Figure 12.—Alpha Beta Market. Front (south) shear wall, looking east. D. F. Moran photograph.

Figure 13.—Alpha Beta Market. Southeast corner of building east of market; brick veneer broke loose from its stud wall backing. Wheeler & Gray phoograph.

Figure 14.—Alpha Beta Market. Front (west) curtain wall. D. F. Moran photograph.

Figure 15.—Alpha Beta Market. New gunite shear wall replacing damaged brick wall south of building at line B. Wheeler & Gray photograph.

crete walls. One of the finished walls is shown in figure 15. The second one was about to be built and figure 16 shows preparation of the area to be occupied by the new wall.

Wood rafters, which had extended to the back (north) wall, were cut back about 12 inches and a new section of rafter, 5 feet long, was nailed to the original member (fig. 17). Steel straps bolted to the wood rafters and to the wall through the wood ledgers were added (fig. 18). Damaged wood ledgers and

Figure 16.—Alpha Beta Market. New gunite wall will replace damaged brick shear wall that was removed from this area, south of building at line B. Wheeler & Gray photograph.

Figure 17.—Alpha Beta Market. Damaged ends of wood rafters at north wall were cut back, and 5-foot sections of new rafters were nailed to the existing members. Wheeler & Gray photograph.

Figure 18.—Alpha Beta Market. New steel straps bolted to north wall and to wood rafters. Wheeler & Gray photograph.

boundary plywood were replaced and renailed.

Additional knee braces between steel pipe columns at line 5 and the bottom flange of tapered steel girders were added (fig. 19). It is assumed that braces are intended to improve lateral stability of the building.

Pipe columns on line 4, leaning about 4 inches toward the east, were straightened during repairs. All cracked concrete walls were to be pressure injected with epoxy compound, and concrete pilasters were to be gunited after proper preparation of the affected area (fig. 9). Completion of structural work was to be followed by repair of architectural items such as finishes, ceilings, fixtures, and others.

Repair costs amounted to approximately $136,000, which represents about 35 percent of the value of the building.

CONCLUSIONS

The effect of differential movements between two structures of different stiffnesses was clearly indicated by damage sustained to pilasters at connections with tapered steel girders from lower roofs (figs. 8, 9, and 10). Also, cracking of concrete wall was much more pronounced in side walls with adjacent lower buildings than in end walls, where no lower construction existed. As in other buildings investigated and covered in other sections of this volume, the end wall connection to the roof diaphragm proved to be inadequate.

Figure 19.—Alpha Beta Market. New steel knee braces added between steel columns and tapered steel girder to improve lateral stability of building at line 5. Wheeler & Gray photograph.

RECOMMENDATIONS

1 Adequate separation should be provided between two buildings or areas of different rigidities; otherwise, full allowance should be made for the effects of the different periods of vibration.

2 Anchorage of brick veneer to wood stud walls should be improved.

Boys Market (13)

2040 Glenoaks Boulevard, San Fernando

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WHEELER & GRAY

Consulting Engineers

Los Angeles, Calif.

GENERAL DESCRIPTION

The Boys Market, which was demolished after the earthquake, was a one-story building with a wood frame roof and masonry walls, typical of many market buildings in the area. Part or all of two walls were open with extensive glass areas. This building was basically 181 by 212 feet in plan and 18 feet in height (fig. 1). An adjacent shop building encroached into the northeast corner of the building (54 by 60 ft).

The roof was constructed with glued laminated wood beams supporting wood purlins at 8 inches on center. Due to the irregular spacing of the pipe columns supporting the roof, wide-flange and glued laminated girders were required to carry the loads from roof beams to the columns (fig. 3, details 7 and 8). The roof sheathing was ½-inch plywood, framed into 2- by 4-inch rafters at 24 inches on center spanning between roof purlins. Glued laminated roof beams cantilevered 8 feet to form a canopy beyond the north and west walls and returned 20 feet along the south wall (fig. 2, section 4–4 and fig. 3, detail 6). A lower metal deck roof on steel framing covers a loading dock at the southeast corner of the building. A small mezzanine located over the restroom and locker area (fig. 1) was constructed with ¾-inch plywood floor sheathing on wood joists, which in turn were supported by steel beams and columns. Refer to figure 1 for the market framing plan indicating the roof construction.

The enclosing building walls were 8½-inch-thick reinforced brick, except for the west and portions of the north and south walls, which were glass (fig. 1). The reinforcing in the brick walls was No. 4 reinforcing bars at 24 inches on center, vertically and horizontally, which was the minimum required by the building code.

The site foundation investigation indicates that the soils beneath the site, to a depth of 35 feet, con-

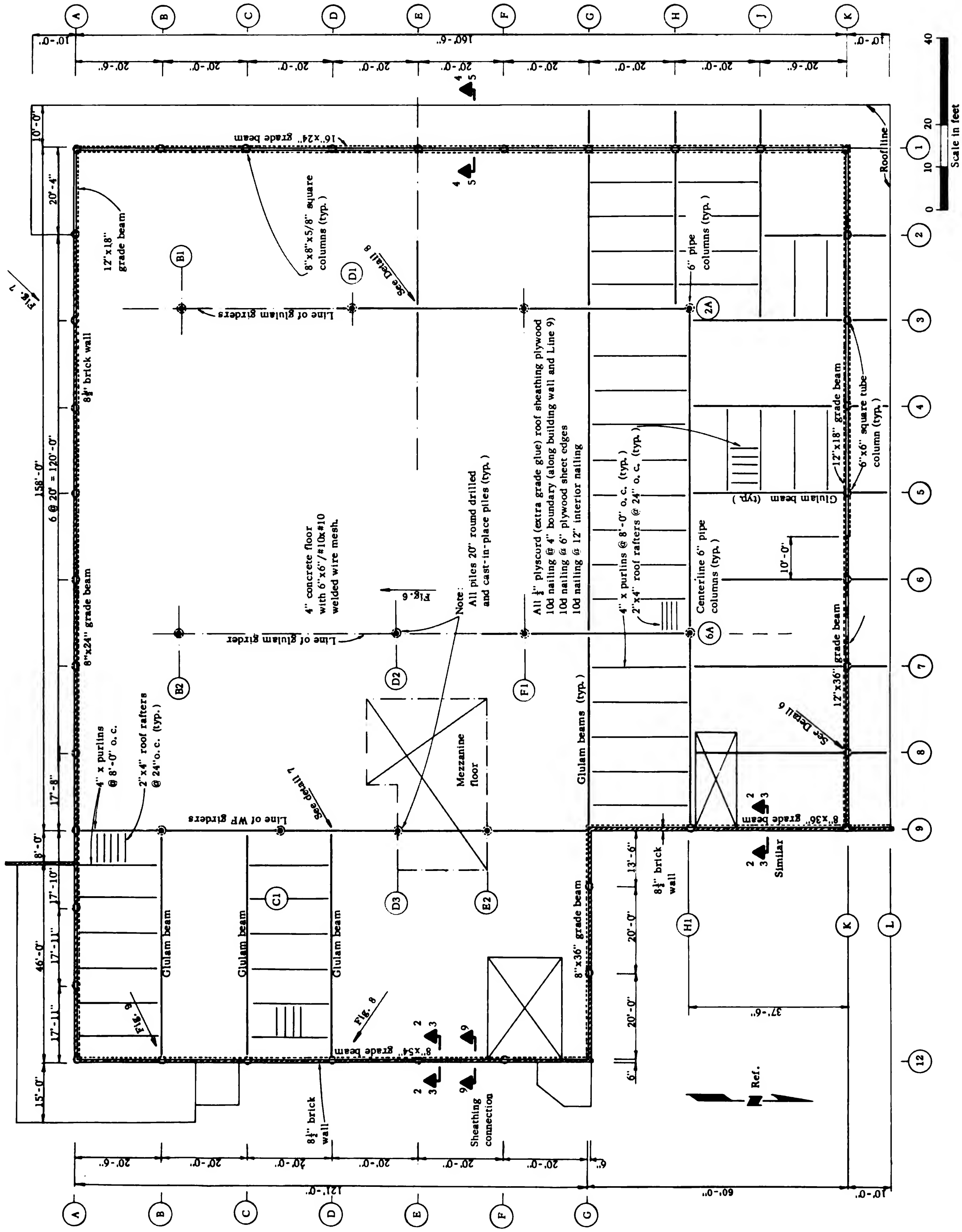

Figure 1.—Boys Market. Foundation and roof framing plan.

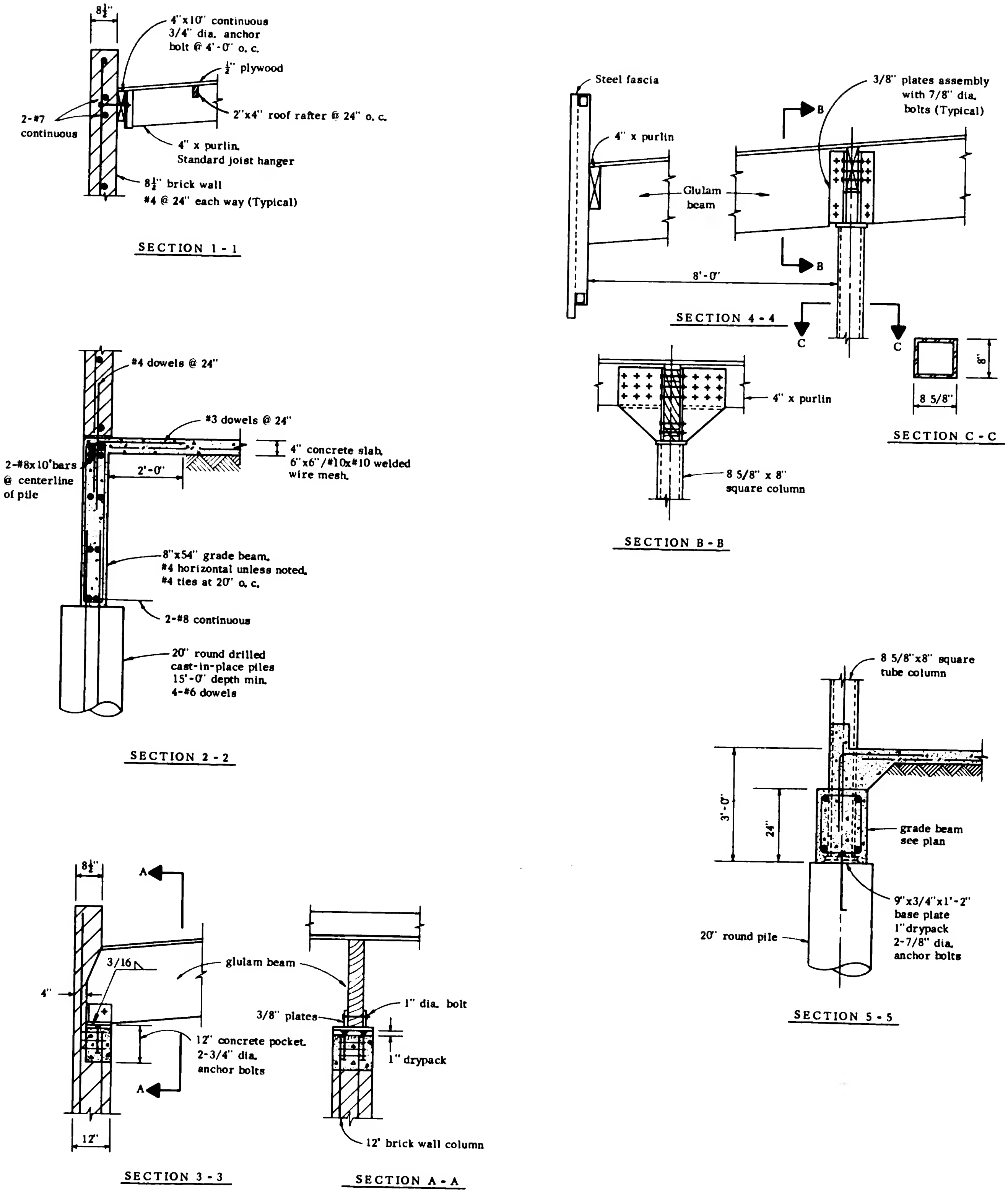

Figure 2.—Boys Market. Sections.

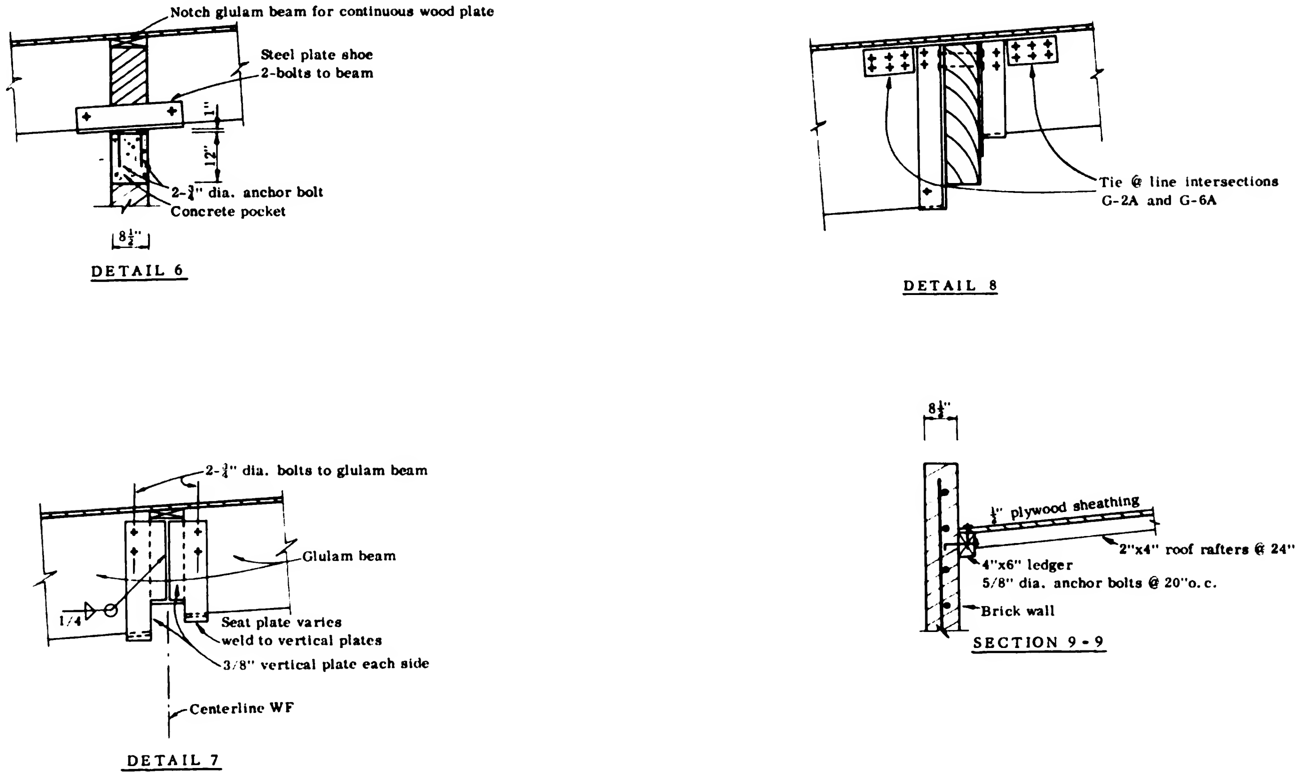

Figure 3.—Boys Market. Section and details.

sist of silt and silty sand with minor layers of sand found in some locations. The upper natural soils are relatively weak and compressible and very adversely affected by moisture.

Twenty-inch-diameter drilled and cast-in-place friction piles were used for building support (fig. 1; fig. 2, sections 2–2 and 5–5). For resisting lateral loads at ground level, drilled piles and the frictional resistance between the floor slabs and supporting soils were used. Figure 1 indicates typical foundation construction. The floor slab was a 4-inch slab on grade and reinforced with wire mesh. The piles had a minimum length of 15 feet and were reinforced with a minimum of four No. 6 reinforcing bars. All concrete had a minimum ultimate compressive strength of 2,000 psi at 28 days, except the piles which had a minimum strength of 2,500 psi.

A box system, as defined by the 1958 Uniform Building Code, was used in design to resist seismic-induced forces. The seismic force was determined using a horizontal force factor of 13.3 percent of gravity. The plywood roof sheathing acted as a horizontal diaphragm delivering loads to the exterior walls, which in turn acted as vertical resisting elements. Sections 1–1 and 3–3 on figure 2 and section 9–9 on figure 3 indicate typical connections between the roof construction and the walls. The west wall supported the roof diaphragm laterally through the use of steel columns cantilevered up from foundation grade beams. All other walls resisted the load through shear in the 8½-inch brick. The roof is tied in two directions to the masonry walls at the reentrant corner G–9 (fig. 1) by the glued laminated beam and the wide-flange girder meeting there. These members (in addition to serving their purpose as beams and girders) function as struts, which transmit lateral loads from the roof diaphragm to the walls. The building appeared to have been designed for the lateral forces required by code.

EARTHQUAKE DAMAGE

This building site is located on the Sylmar segment of the San Fernando fault zone. It is located on the extreme west end of the surface expressions of differential movements in this segment. Vertical and horizontal permanent differential movements, in addition to both compressive and tensile forces, were active in the area. Across the disturbance zone and within a few hundred yards of the building site, a compressional shortening of 0.3 m and a differ-

Figure 4.—Boys Market site. Looking north, showing in addition to the market, the adjacent shops building and bowling center. Robert E. Wallace, USGS, photograph.

ential horizontal movement of 1 m were encountered. In the parking lot adjacent to the northwest corner of the building, a permanent rise of 5 feet was recorded in the surface elevation. It is estimated that the floor of the market may have lifted between 4 and 5 feet, tilting slightly down to the southeast. For detailed information on ground movement, see "Planetable Survey of Parking Lot Damaged by San Fernando Earthquake" in Volume III.

Figure 4 is an aerial view of the site of Boys Market. On the right of the photograph is a bowling center with a shops building between the center and the market. Surface breaks from fault movement can be discerned easily. Note that, in addition to damage to

Figure 5.—Southwest corner of bowling center. Looking west, approximately 100 feet east of Boys Market. Area ground movement. Note offset in curb in near background. LeRoy Crandall & Associates photograph.

Figure 6.—Boys Market. Typical north-south crack in building floor. LeRoy Crandall & Associates photograph.

Figure 7.—Boys Market, looking toward location of south wall. Note trunk of auto in center of photograph under fallen wall. LeRoy Crandall & Associates photograph.

the market, the shops building lost its east wall and part of its south wall. Large differential ground movement took place in the area between the bowling center and the shops. Figure 5 shows the southwest corner of the bowling center, looking west toward the market, and indicates one type of movement.

The market building floor was cracked badly. One of the typical cracks is shown in figure 6. It was reported that the open cracks were in the range of 4 inches in width, with differential vertical displacement in the same range at several locations. All observed floor cracks were thought to be tension cracks.

The masonry portion of the south wall, west of the loading dock, lost its lateral support during the earthquake and fell. The first bay of roof framing also fell to the ground. Figure 7 is looking toward the location of the south wall. Although the roof beams, rafters, and sheathings separated from the east wall, the wall remained in place. Figures 8 and 9 show how the beam seats along the east wall were torn apart. It was noted that approximately one-half of the ledgers bolted to the wall were ripped down.

The market, which had an estimated replacement value of $420,000, was declared a total loss.

Figure 8.—Boys Market. East wall, showing roof beams torn from their seats. LeRoy Crandall & Associates photograph.

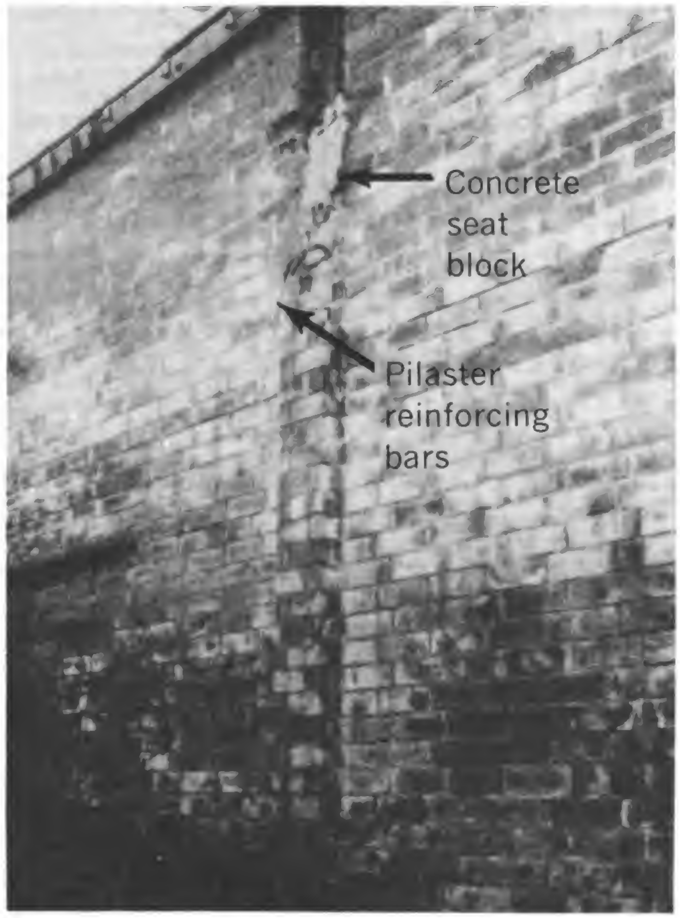

Figure 9.—Boys Market. Closeup of roof beam seat. Note broken concrete bearing block and reinforcing bars which are bent down the wall. LeRoy Crandall & Associates photograph.

Behavior of Joist Anchors Versus Wood Ledgers

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125 Conclusions and Recommendations

EZIO BRIASCO

Los Angeles City Department of Building and Safety
Los Angeles, Calif.

This report compares the performance of single-story industrial and commercial buildings with strap-type roof joist anchors at exterior walls and buildings constructed with 4-inch wood ledgers with plywood roof sheathing nailed on top of the ledger to anchor walls to the roof.

Details 1 through 4 of figure 1 show typical strap joist anchor installations. Detail 6 of figure 1 shows a typical roof-to-wall connection using a 4-inch wood ledger.

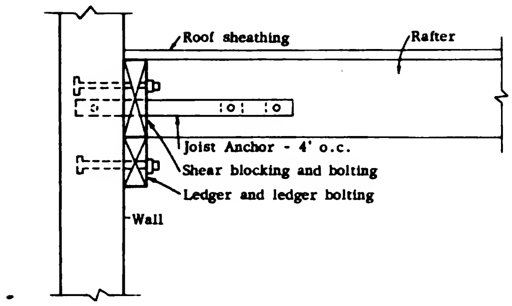
The exterior walls of these buildings are of masonry or concrete tilt-up construction, and the roofs are framed as plywood diaphragms. Tables 1 and 2 and figure 2, respectively, are lists of buildings and a map of the Sylmar industrial tract.

POSTEARTHQUAKE SURVEY

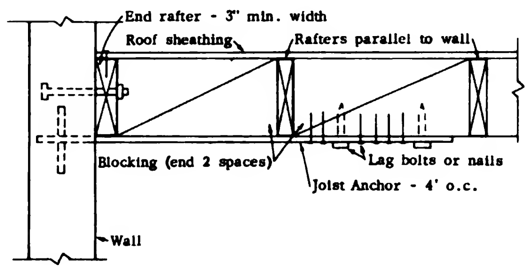
After the earthquake, a number of buildings in this area that were constructed with joist anchors were inspected by building department officials (table 2). The exterior walls of these buildings were found to be tight to the roofs without any apparent separation. At the time of this writing, no building constructed with joist anchors is known to have sustained separation between roof and walls.

As indicated in table 1, many buildings constructed with 4-inch wood ledgers and plywood failed. Roofs separated from walls and portions of many roofs fell down. In the majority of cases nails pulled out of the plywood, but, in some cases, the plywood pulled nails out of the ledger, or the ledger split in cross-grain bending and broke at the bolt line.

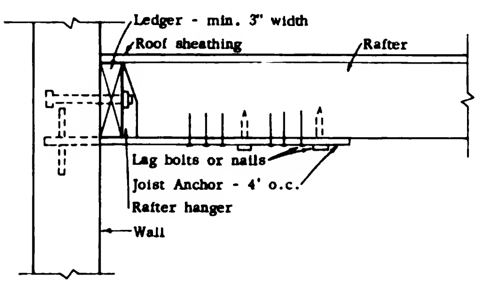
It should be mentioned that the human element can play an important role in the lateral strength of this type of roof-to-wall connection. The lap of the plywood sheets on top of the wood wall ledgers, the spacing of the plywood nailing with regard to edge distance both in the plywood and the wood ledgers,



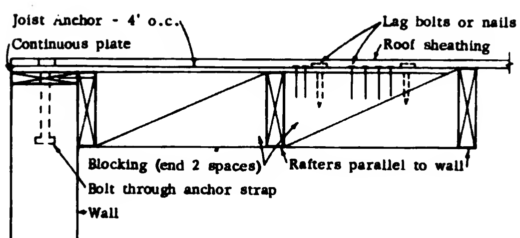
DETAIL 1 Joist Anchor Installation



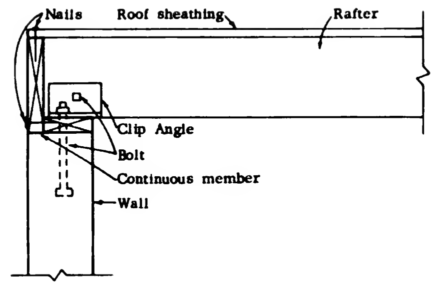
DETAIL 2 Joist Anchor Installation



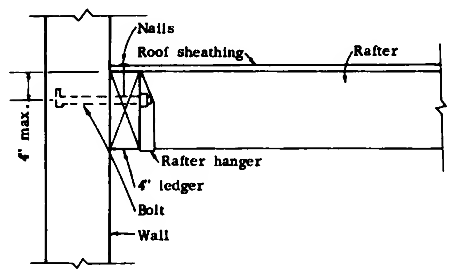
DETAIL 3 Joist Anchor Installation



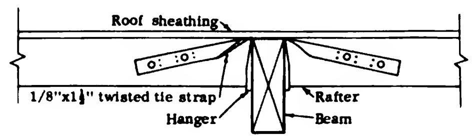
DETAIL 4 Joist Anchor Installation



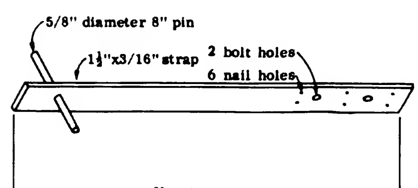
DETAIL 5 Wall to Roof Anchorage



DETAIL 6 4" Wood Ledger and Roof Sheathing Anchorage



DETAIL 7 Continuity Tie Strap



DETAIL 8 Typical Joist Anchor

Figure 1.—Typical joist anchor installations (details 1 through 4) and other roof-to-wall connections (details 5 and 6). A continuity tie strap (detail 7) and a typical joist anchor (detail 8) are also shown.

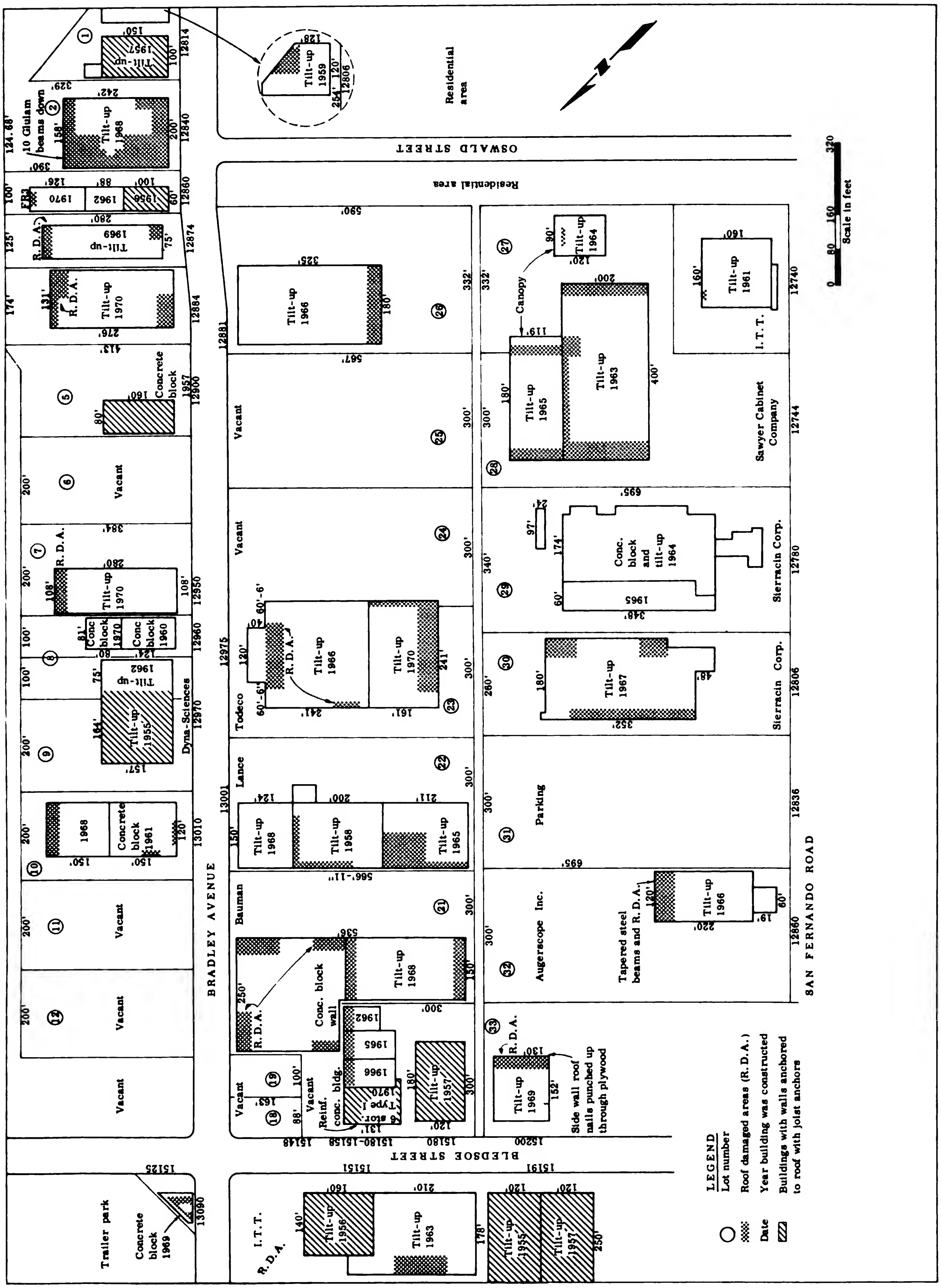


Figure 2.—Map of Sylmar industrial tract showing buildings in which roofs collapsed or separated from walls.

Table 1.—Plywood roofs with 4-inch ledgers without joist anchors in Sylmar industrial tract

[Roof areas down or detached from ledger]

Sylmar address	Year constructed	Size	Type
		Feet (inches)	
15148 Bledsoe Street....	1968	260 by 536...	Tilt-up.
15151 Bledsoe Street....	1963	210 by 178 (add).	Do.
15180 Bledsoe Street....	1966	64(8) by 117(8).	Do.
Do...............	1965	....do......	Do.
15125 Bledsoe Street....	1969	68 by 82.....	Concrete block.
15200 Bledsoe Street....	1969	130 by 152...	Tilt-up.
12806 Bradley Avenue..	1959	120 by 150...	Do.
12840 Bradley Avenue..	1968	158 by 242...	Do.
12860 Bradley Avenue..	1970	60 by 126(8) (add).	Concrete block.
12874 Bradley Avenue..	1969	75 by 280....	Tilt-up.
12884 Bradley Avenue..	1970	131(6) by 276	Do.
12950 Bradley Avenue..	1970	108 by 280...	Do.
12881 Bradley Avenue..	1966	180 by 325...	Do.
12975 Bradley Avenue..	1966	241 by 281...	Do.
Do...............	1970	241 by 161...	Do.
13001 Bradley Avenue..	1958	150 by 200...	Do.
Do...............	1965	150 by 211...	Do.
Do...............	1968	150 by 124...	Do.
13010 Bradley Avenue..	1961	120 by 150...	Concrete block.
Do...............	1968	120 by 150 (add).	Do.

the penetration of the nail head into the plywood surface, and the vertical placement of the ledger anchor bolts within the ledger all may have a pronounced effect on the resistance of the joint to pull apart under earthquake forces. It also may be that diaphragm nail values need a reappraisal in light of these factors. Small-scale tests may be indicated as a means of improving this connection.

Tying buildings together from exterior wall to exterior wall is another problem area. (The Los Angeles City Building Code requires that buildings be tied continuously from exterior wall to exterior wall at roof and floorlines; however, roof sheathing may be used to satisfy this requirement at the time of this writing.) It was found that where rafters or purlins were supported in hangers butting into beams, the rafters or purlins pulled away from the beams in some cases. Even where the plywood was staggered across the beams the plywood broke away. It could be expected that this type of failure would be more common in the future if better anchorage is provided at the walls. The walls probably will hold to the roof and there will be greater tension stress on the interior panels of the roof.

An example of this type of failure was at 12366 Montero Avenue. This is a 101- by 160-foot tilt-up building, constructed in 1966, with a plywood panelized roof. The 4-inch-wide roof purlins at 8 feet on centers were connected to the northeast and southwest walls with heavy clip angles and bolts. The purlins and roof held to the exterior walls; however, in several areas the purlins pulled away from the main interior roof beams and dropped, imposing a serious safety hazard. The plywood nails pulled out of the main beams and also broke out of the sides of the 2- by 4-inch rafters, even where the plywood was staggered across the main beams.

Table 2.—Buildings with joist anchors in Sylmar industrial tract

Sylmar address	Year constructed	Building size	Roof sheathing	Exterior walls	Roof beams [1]	Rafter connection at beams	Joist anchor connection	Remarks
		Feet						
15151 Bledsoe Street......	1957	120 by 180.	Plywood..	Tilt-up...	T.S.G.	Hanger...	Bolted....	Roof tight to tilt-up walls.
15180 Bledsoe Street......	1956	140 by 160.	....	do....	G.L.B.	....do..............		No reported roof damage; roof concealed by roof insulation.
15191 Bledsoe Street......	1955	120 by 150.	....	do..................................				Walls held to roof.
Do.................	1957	120 by 150 (add).	....	do..................................				Do.
12814 Bradley Avenue.....	1957	100 by 149.	....	do..................................				Inspection reports no roof damage.
12860 Bradley Avenue.....	1956	60 by 100.	1-in. diagonal sheath.	Concrete block.	T.S.G.	Lapped over the beam.	Nailed...	Nails in joist anchors worked loose a little; roof tight to exterior walls.
12900 Bradley Avenue	1957	80 by 160.	....	do..................................				No reported roof damage.
12970 Bradley Avenue.....	1955	157 by 164.	Plywood..	Tilt-up...	T.S.G.	Hanger...	Bolted....	Roof tight to exterior walls.

[1] T.S.G. = Tapered steel girder.
G.L.B. = Glued laminated beam.

CONCLUSIONS AND RECOMMENDATIONS

It is concluded from this survey that the joist anchor is one solution to the problem. It is recommended that the roof-to-wall anchorage for this type of building be given further study. Also, considerations to tie buildings at the roof and floorlines continuously from exterior wall to exterior wall should be studied.

The Los Angeles City Building Code was changed on August 10, 1957, to allow the use of 4-inch wood ledgers with plywood nailed to the top of the ledger, in lieu of joist anchors, to anchor walls to roofs. Since the earthquake, an ordinance change has been enacted that no longer permits wood ledgers in lieu of joist anchor-type connections.

Wood Roof and Masonry Wall Buildings— Summary, Conclusions, and Recommendations

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SUBCOMMITTEE ON BUILDINGS

NOAA/EERI Earthquake Investigation Committee

The preceding 13 building reports have presented only those conclusions and recommendations that were unique to the individual buildings studied. Following are conclusions and recommendations that apply to two or more of the wood roof and masonry (including concrete) wall buildings covered in the preceding section. Because of certain similarities in building materials and types of construction, this group of one-story light industrial and commercial buildings sustained similar patterns of damage. Although the intensity of ground movement and soil conditions varied somewhat among building sites, and the degree of damage varied considerably, the studies of these buildings resulted in a strikingly large number of similar conclusions. The paper titled "Behavior of Joist Anchors Versus Wood Ledgers" points out the superior behavior of the buildings that had steel joist anchors tying the walls to the roof system.

CONCLUSIONS

1 One of the most common occurrences was the separation of the roof from the building end walls, i.e., walls parallel to roof girders. The perimeter sheathing nails often pulled through the plywood sheathing or out of the wall ledgers. In other cases the ledgers failed in cross-grain bending and became completely separated from the wall. The ledger-to-purlin or rafter connection was commonly little more than a commercially available metal seat intended only for support of vertical loads. Thus, when the walls moved outward or collapsed, the adjacent portions of the roof generally collapsed. The seriousness of this mode of failure in regard to life safety is that it is a sudden or "brittle" type of failure with little warning. The individual reports clearly indicate where this type of failure has occurred.

Analysis of the probable actual forces acting on many wall-to-roof connections, assuming a 30 percent of gravity horizontal acceleration force level, indicates a force of about 300 pounds per lineal foot acting perpendicular to the wall. Nails spaced at 4 inches on center produce a load of about 100 pounds per nail. Ultimate strengths of 10d nails in ½-inch plywood have been tested statically at 500 to 600 pounds per nail where nail spacing and edge distance were not factors.

It seems reasonable that an edge distance of only ⅜ inch, which was generally used on these buildings, could reduce the strength and result in failure. It is significant that similar wall-to-diaphragm separations did not occur in buildings where steel strap-type joist anchors were used to anchor the walls to the roof framing.

2 The roof-to-side wall connection, i.e., walls perpendicular to and supporting main roof girders, performed much better than the roof-to-end wall connections. Numerous buildings sustained damage in this area, but no collapses were observed in the buildings studied. The stability of the side walls was improved notably by the stiffening effect of built-in pilasters.

In a roof diaphragm, the lateral force unit shears parallel to the end walls are higher than those parallel to the side walls. This is because there is less diaphragm width available adjacent to the short end walls to resist the lateral forces. These higher unit shears probably resulted in initial loosening or working of the nails, leading to subsequent failure.

3 Another very common type of damage was the distress of the roof girder-to-wall connection. Seismic forces acting perpendicular to the walls became concentrated at this connection, and frequent spalling of the tops of pilasters and pulling out of girder seat anchor bolts resulted.

4 Damage occurred at wall corners due to lack of adequate continuity of horizontal wall reinforcing around the corners. Corners suffered vertical cracks and some shattering. No significant difference in behavior between precast concrete, brick, or hollow concrete block walls was noted except where precast concrete connection details were poor.

5 Damage was influenced by permanent ground movements at some locations.

6 Damage frequently occurred at points of discontinuity such as reentrant corners. The cause of this damage generally could be traced to inadequate diaphragm connection details and lack of continuity in collector members.

7 Separations in some roof diaphragms also occurred in the interior areas away from the exterior walls. There is a lack of continuity here, because the purlins are only supported vertically in hangers and the plywood sheathing must act as the tie across the buildings. Separations occurred even where plywood sheets were staggered.

RECOMMENDATIONS

1 The connection between the roof diaphragm and the walls should be improved. Criteria should be developed to provide realistic design force and detail requirements for connecting these elements. Some ductility in the behavior of these connections would be desirable to avoid "brittle" failures. Details should be subjected to realistic simulated earthquake loadings prior to approval.

One apparent effective method of improving the roof-to-wall connections is to use steel joist anchors to anchor the walls to the roof framing members.

2 Stronger connections between main girders and supporting pilasters are recommended. Improvement in the containment of masonry and concrete at the tops of wall pilasters should be studied. A means of better confining seat anchor bolts should be studied. Again, ductile behavior under lateral loads would be desirable.

3 Continuity of wall construction at corners of roof diaphragms should be improved.

4 Where roof diaphragms contain reentrant corners or other forms of irregularity, particular attention should be paid to details and connections.

5 Continuity should be provided completely across the building by tying together the purlins, joists, or other members in addition to the plywood sheathing.

6 It is not economically feasible to design buildings to resist permanent ground displacements. Soil and geologic studies are recommended in order to avoid construction in potentially hazardous areas.

Museum for Antique Cars (14)

15180 Bledsoe Street, Sylmar

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Photographs provided by Los Angeles City Department of Building and Safety, John Shadle, photographer.

WHEELER & GRAY
Consulting Engineers
Los Angeles, Calif.

GENERAL DESCRIPTION

The Museum for Antique Cars is a five-story reinforced concrete building that was under construction on February 9, 1971. The construction had proceeded through the fifth-floor level, and work was progressing above the fifth floor on two concrete stair and elevator towers diagonally opposite each other on the northeast and southwest corners of the building. Most of the fifth floor was to be occupied by a steel frame structure that had not been erected prior to the earthquake. The photograph in figure 1, taken shortly after the earthquake, indicates the progress of construction.

The museum building has a rectangular plan, 80 feet by 131 feet 6 inches, with the fifth floor slightly larger than the other floors having cantilever slabs on two sides. The fifth floor cantilevers 4 feet beyond the north wall and 8 feet beyond the west wall. An elevator tower located on the southwest corner of the building outside the basic building rectangle extends in excess of 34 feet above the upper floor. The tower on the northeast corner of the building extends 26 feet above the fifth floor. A mezzanine is located between the second and third floors and is approximately 26 feet 6 inches wide, extending along the north and west walls. Figures 2 and 3 are representative of the five floor plans of the building, and figures 4 and 5 are typical building cross sections.

This building is within the area known as the Sylmar industrial tract. The tract introduction contains a description of general site conditions. More specific information is included in the site foundation investigation, which indicates a soil profile of 6 to 10 feet of moderately loose, porous, silty sand followed by moderately compact, silty, fine to medium sand to a depth of 11 feet. Below this are compact and firm alternating layers of silty sands, clean sands, and sandy silt to depths of the borings, 50 feet maximum.

Figure 1.—Museum for Antique Cars. Building north wall, with repair of wall cracks in progress.

Concrete pile caps with three drilled and cast-in-place concrete friction piles furnish typical support for the building. Lateral restraint for the caps is provided by the 6-inch concrete "slab-on-grade" first-floor slab, reinforced with No. 4 reinforcing bars at 24 inches on center in each direction. Concrete used in the foundations had a minimum ultimate compressive strength of 3,000 psi at 28 days, while that used for the floor "slab-on-grade" was 2,000 psi.

The typical floor construction consists of 8-inch concrete flat slabs designed to span in two directions and divided into 26- by 26½-foot bays. Around each of the bays are concrete supporting beams between concrete building columns. In the exterior walls, vertical load-carrying beams and columns are cast integrally with the wall construction. No. 4 reinforcing bars, 10 inches on center horizontally and 16 inches on center vertically, are used in the 8-inch-thick concrete exterior walls. The strength of all superstructure concrete had a minimum ultimate compressive strength of 3,000 psi at 28 days. The floor construction, including the beams in the walls, is lightweight concrete, with an oven dry weight not to exceed 110 pcf. All concrete placed above the fifth floor is lightweight concrete, with an oven dry weight not to exceed 90 pcf.

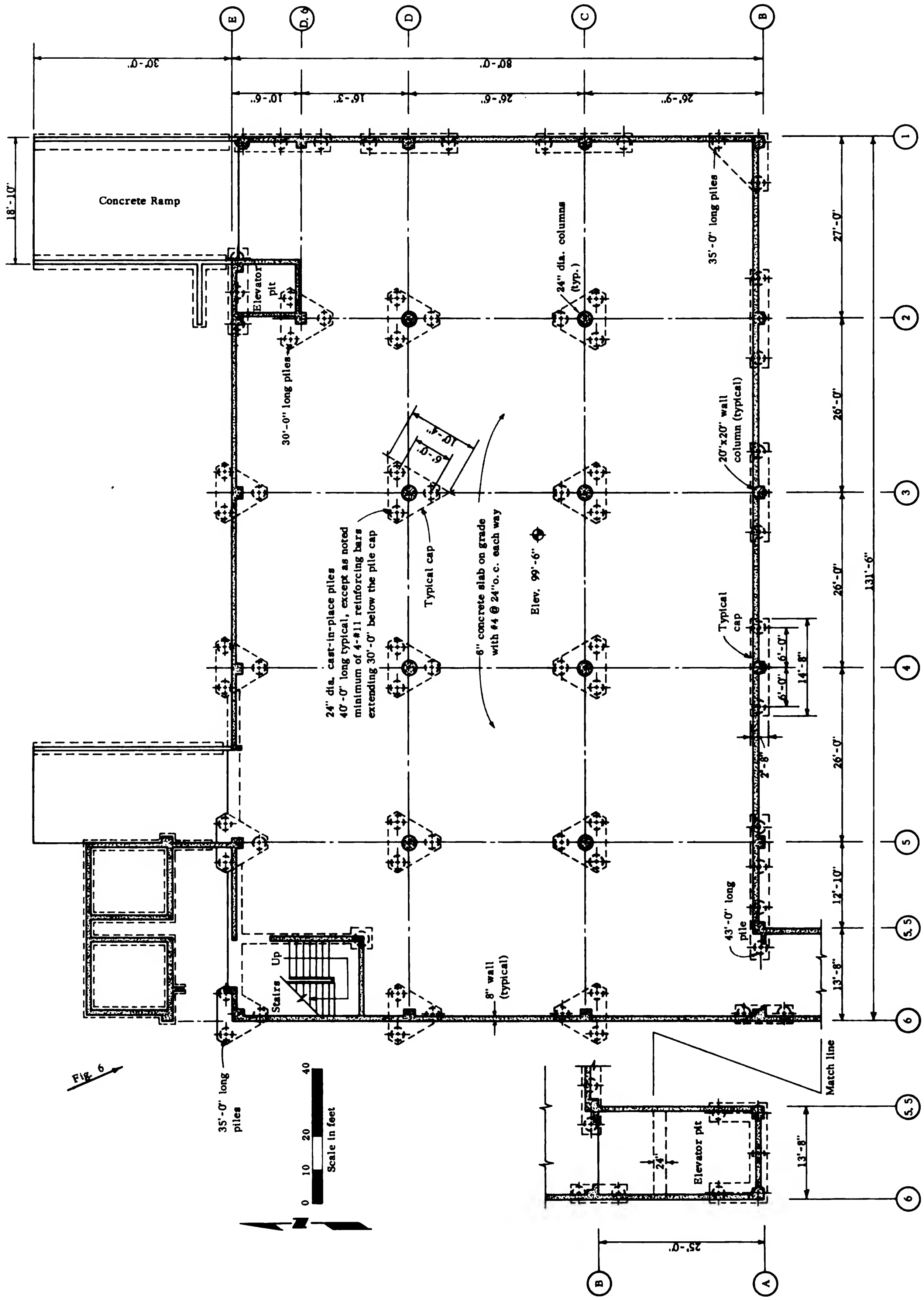

Figure 2.—Museum for Antique Cars. Foundation and first-floor plan.

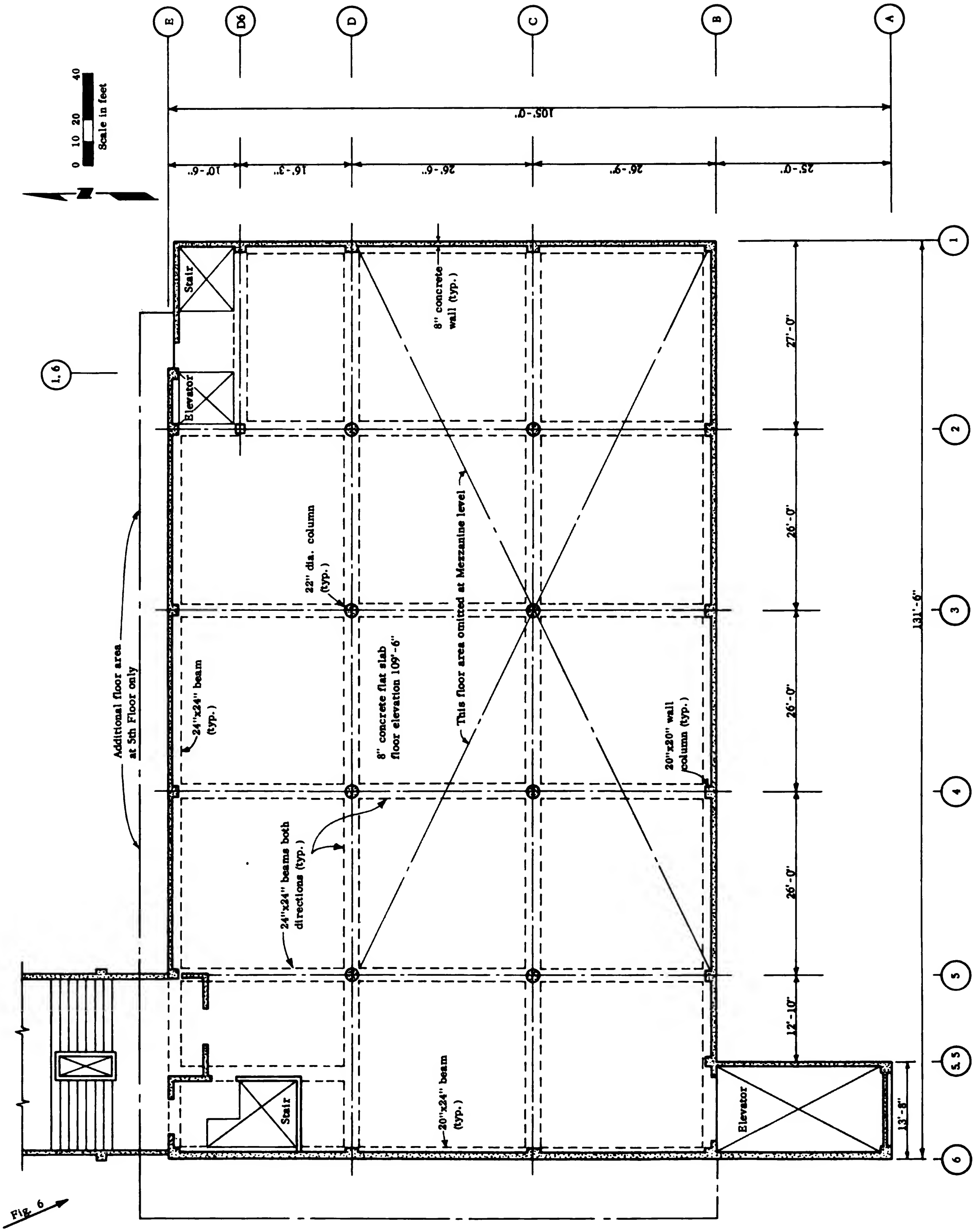

Figure 3.—Museum for Antique Cars. Second-floor framing plan. Mezzanine, third, fourth, and fifth floors are similar.

Although the building was under construction at the time of the earthquake, the structural lateral and vertical load-carrying elements for the concrete five-story portion were completed. All of the damaged concrete had undergone sufficient curing to attain at least its design strength.

EARTHQUAKE DAMAGE

The museum building, rectangular in shape, with virtually solid walls, would not seem to be a building susceptible to damage from seismic action. A cursory inspection just after the earthquake, in fact, did not reveal the true extent of the damage. However, in a more thorough inspection, it was found that the upper portions of this building had sustained permanent differential displacement in respect to the lower portions. The displacement occurred when the major exterior walls failed in shear at the horizontal construction joint just above the first floor or the mezzanine floor, or both. Probable causes for this failure are discussed later in this report.

Except for the south wall, the walls fractured around the building at the first-floor line. The fracture line in the south wall followed the horizontal construction joint along the mezzanine-floor level until it approached the elevator tower at the west end, where the fracture proceeded diagonally to the first floor. The east wall had a failure along the mezzanine-floor level in addition to the one at the first floor. The two walls that failed at the mezzanine-floor level were the two clear height walls that were not laterally supported by the mezzanine floor.

In checking the building it was found that the north wall moved differentially ½ inch east, and near the east end of the wall ½ inch north. The south wall moved ½ inch west. The east and west walls showed no differential movement, but may have been twisted or warped slightly to match the displacement of the other walls. Figure 6 shows the wall elevations with the fracture lines and indicates the building displacement.

On the faces of the walls where differential movement took place, spalling was prevalent along the failure line. The spalling is apparent in figures 7 and 8. The typical No. 4 vertical wall reinforcing bars along the horizontal construction joint in the north and south walls were broken; many in the south wall were broken in two places. The broken bars were offset about ½ inch horizontally, as was the wall, but impressions of the bars in the concrete in the joint indicated temporary differential movements of approximately 1½ inches. The typical wall reinforcing bars may be seen in figure 9. The heavier column bars in the wall columns were not broken as the wall bars were, but were kinked badly and exposed. It should be noted that it is common engineering practice to design the concrete to resist the entire shear force and not to use the wall reinforcing for this purpose.

Each of the exterior walls had typical diagonal shear cracks, predominantly below the mezzanine floor. The cracks varied in width, the maximum being about ¾ inch. The interior concrete walls around the elevator shaft and the stairwell between the first and third floors were cracked, and the lintel beams over doors were also cracked and spalled.

The ground action was not apparent by surface indications. The interior floor slab opposite the east-west elevator wall was cracked and raised because of a slight compression override.

REPAIRS

The structure has been restored to a condition that is well in excess of building code requirements and as close as possible to its original condition. Estimates for the repair of the damage approximated $100,000, which is about 15 percent of the replacement cost of the structure at the time of the earthquake.

The fractures along the slippage or shear planes at the horizontal construction joints were repaired by developing keys across the joints. The keys were made by drilling interlocking cores through the walls. The cores were centered vertically on the horizontal construction joints and located at the vertical wall reinforcing steel. Typical keys are shown in figures 9 and 10. The horizontal area of the keys is about one-half the total horizontal area of the wall. Drypack was placed to develop the required shear across the plane. In areas where the typical wall steel had broken, the bars were welded together to reestablish continuity, with pieces of reinforcing bar or steel flat plate used as splices.

Areas of spalled columns, beams, or walls were cleaned to solid concrete and replaced. Two methods were used to restore the reinforcing where the vertical column reinforcing bars were kinked. In the

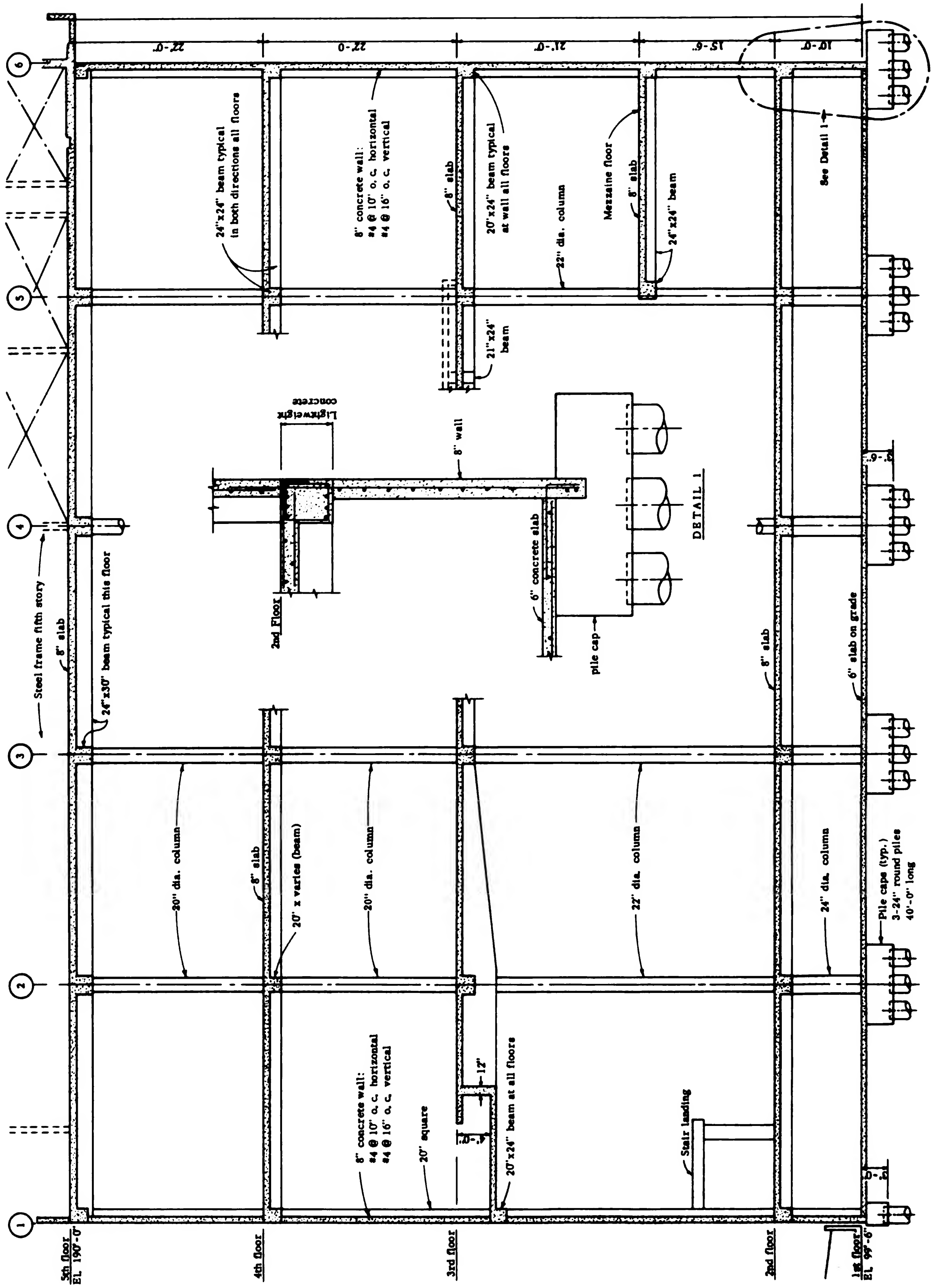

Figure 4.—Museum for Antique Cars. Longitudinal section, looking south.

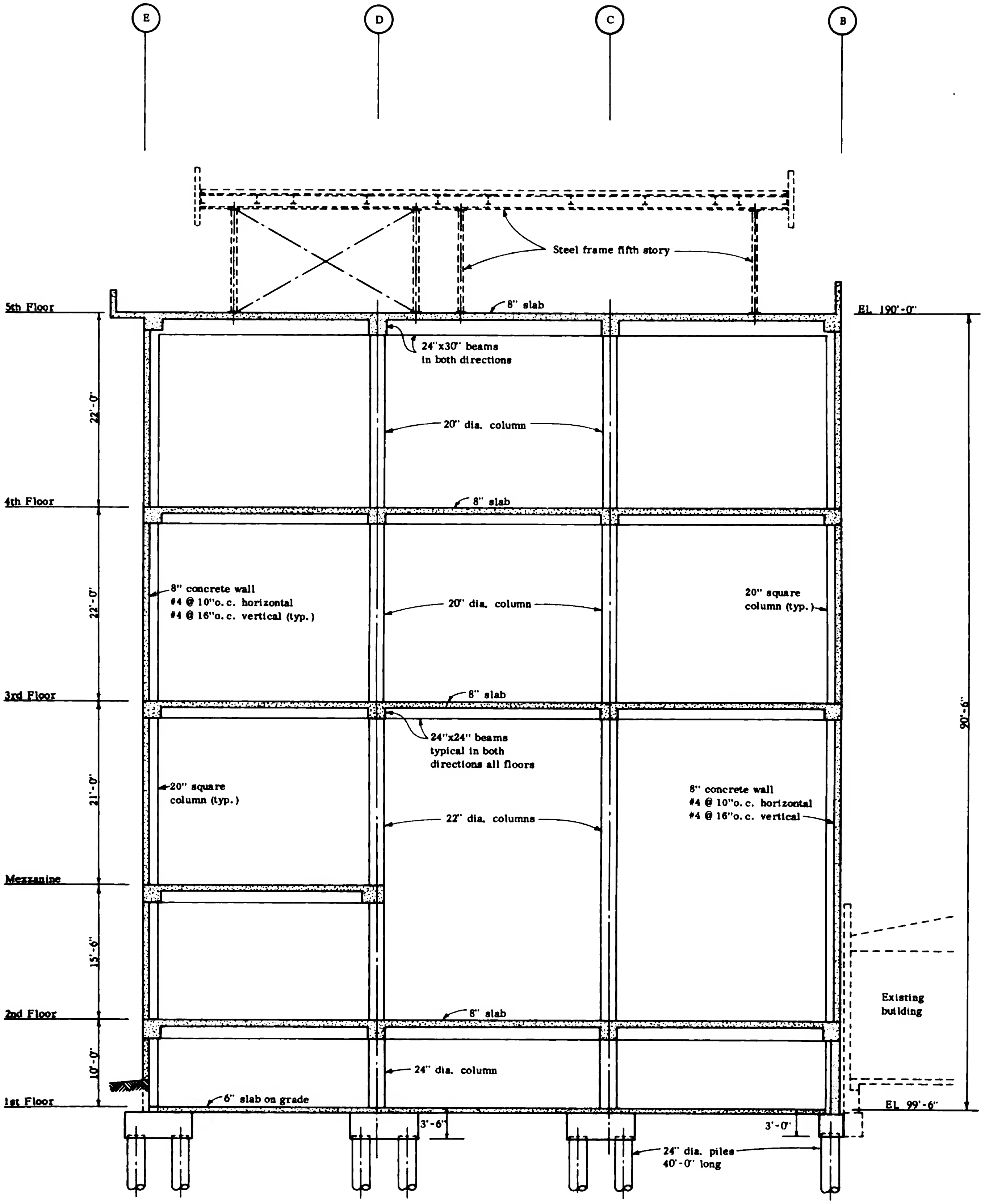

Figure 5.—Museum for Antique Cars. Transverse section, looking east.

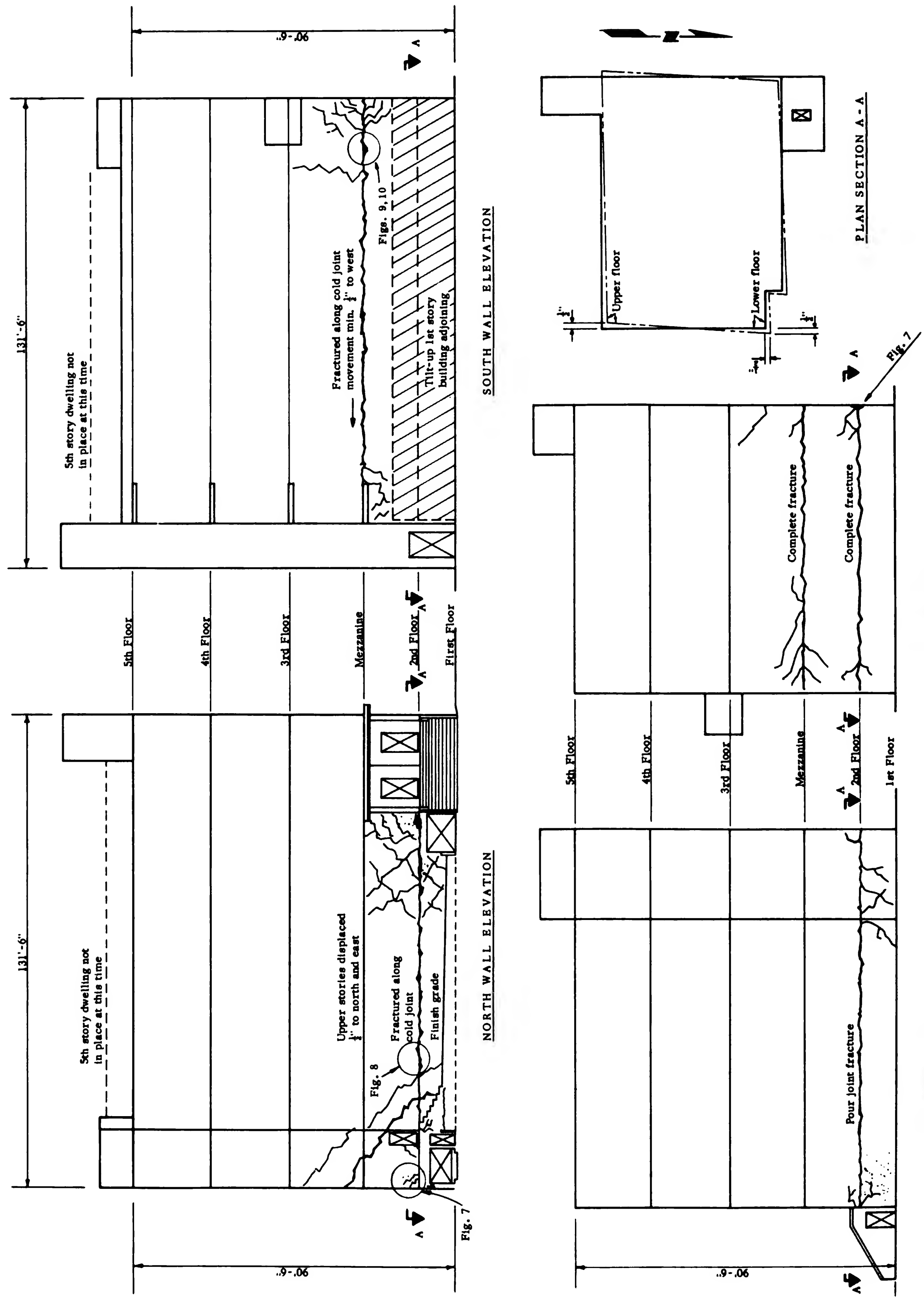

Figure 6.—Museum for Antique Cars. Elevations.

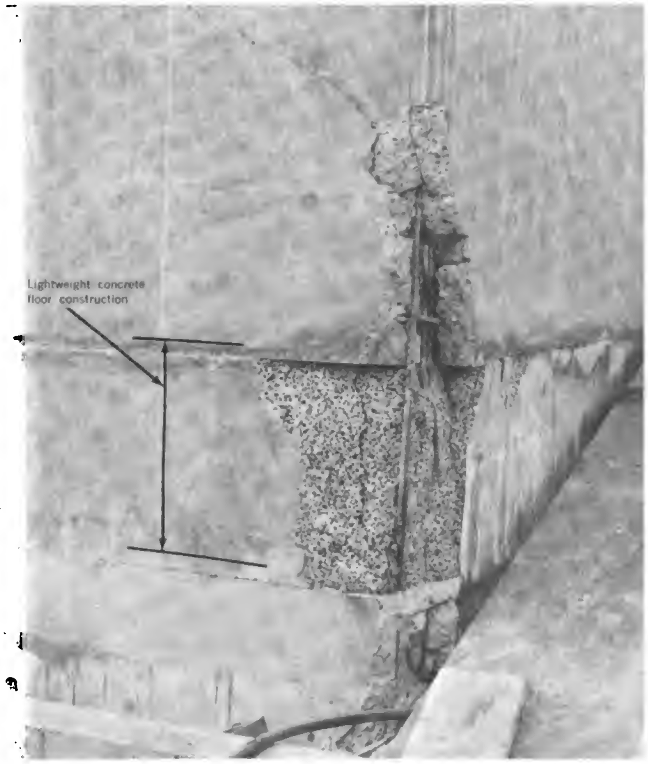

Figure 7.—Museum for Antique Cars. Exterior northeast corner of museum building at first floor. Note lightweight concrete from slab extending through wall.

worst conditions, the kinked area of the bars was cut out and the section replaced using full stress welds. In the other locations lapped splice bars were added. Enough concrete was removed to provide an adequate lap between the new splice bar and the existing bar, both above and below the bent section.

While all visible wall cracks were epoxy grouted, the epoxy grouting was not considered in the repair design of the walls. The walls that were added and replaced were assumed to provide the required lateral support. Two sections in each of the north and south walls were replaced above the second floor. The concrete in these wall panels was removed, preserving the existing wall reinforcing. New dowels were set around the panel edges and grouted. An additional mat of No. 4 reinforcing bars placed diagonally at 45° in each direction was provided for additional shear capacity. Gunite concrete was used to close the wall sections. Figure 11 shows one of the replaced wall panels prior to the placing of the gunite concrete. In the first-floor level where space was not critical, several new wall sections were added adjacent to the damaged walls. After the wall reinforcing steel was placed and dowels were grouted, the new

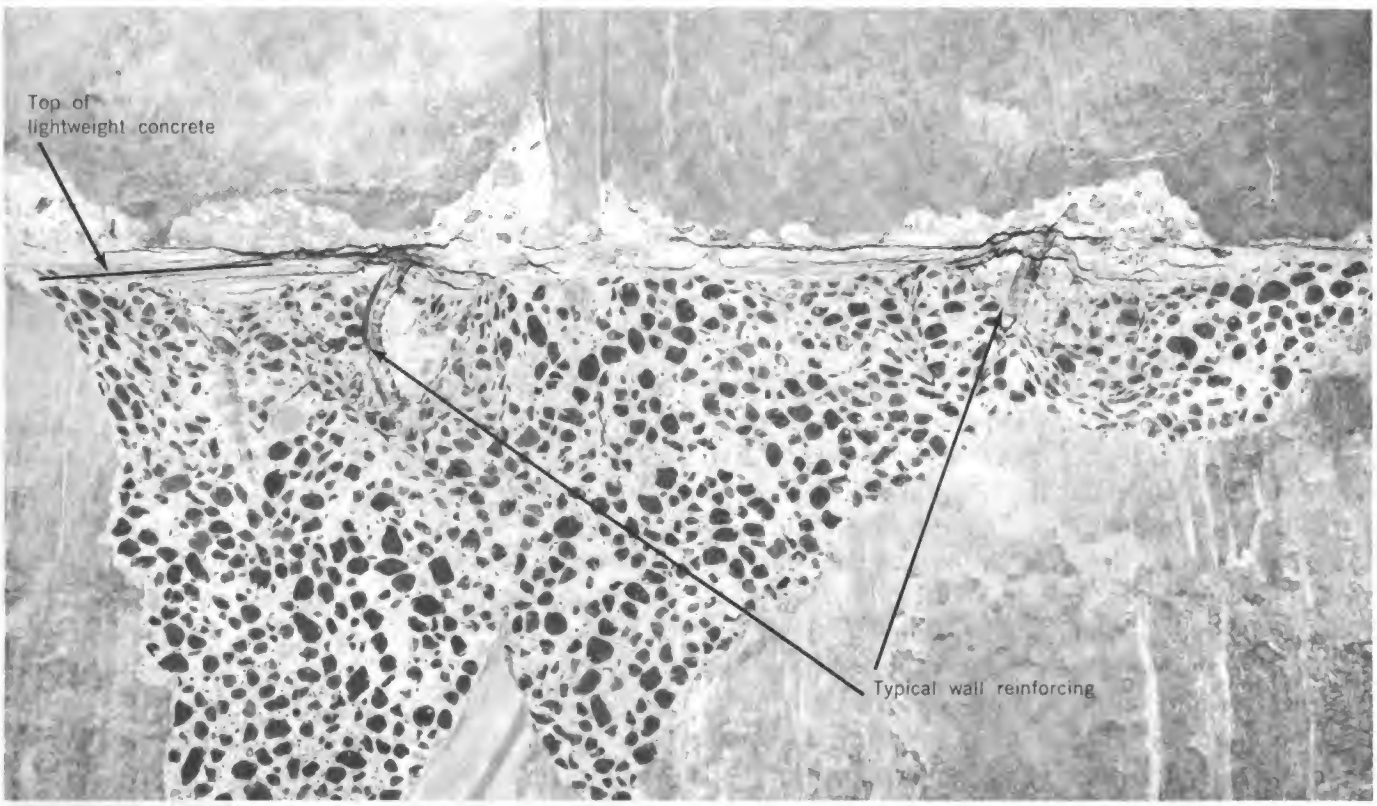

Figure 8.—Museum for Antique Cars. Fracture joint at first floor in north wall.

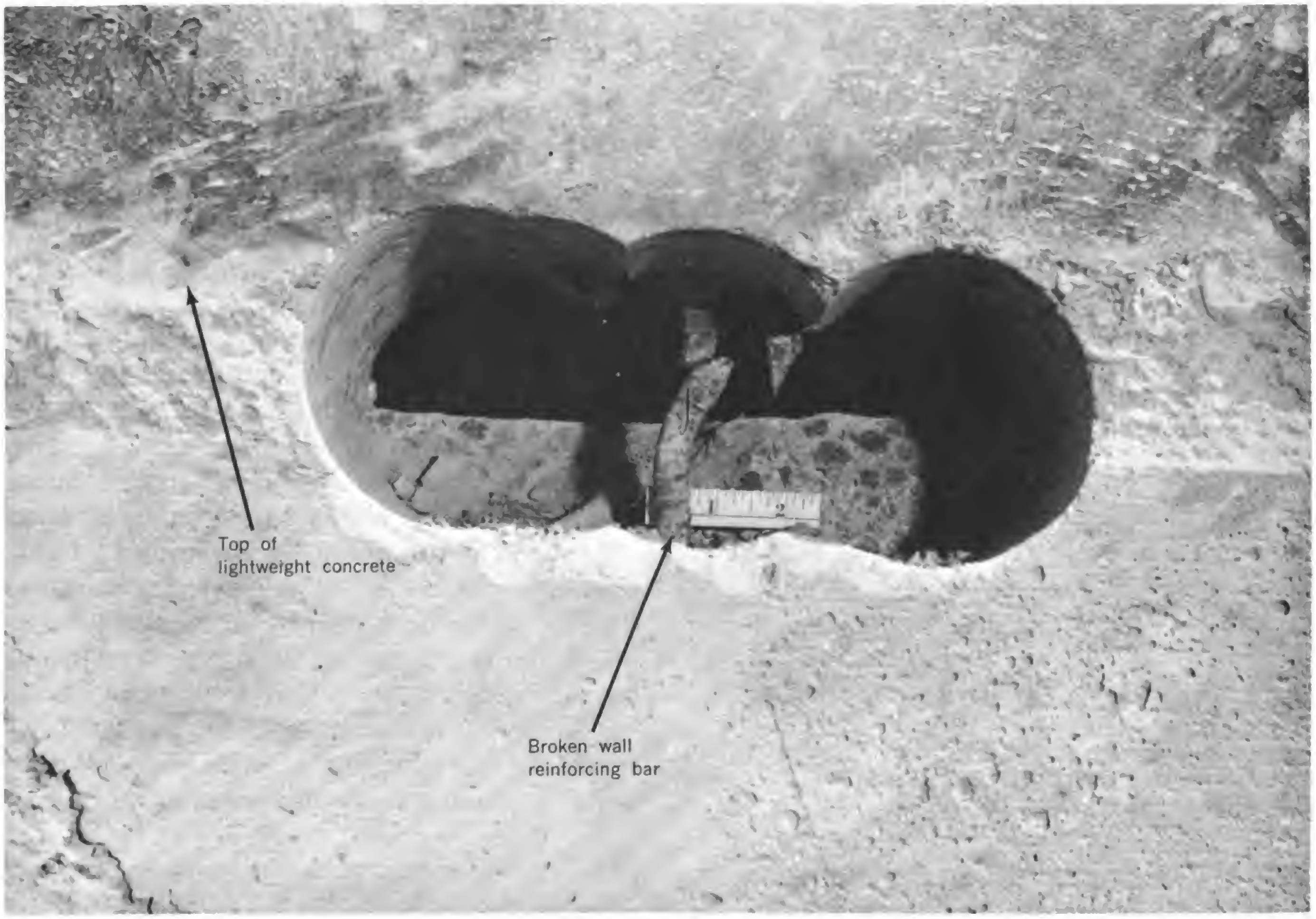

Figure 9.—Museum for Antique Cars. Typical wall reinforcing bars at first floor in north wall. Note offset of upper portion of bar to right of lower portion.

walls were placed using gunite concrete, with the damaged wall acting as forms for placement.

CONCLUSIONS

This building obviously experienced lateral accelerations far in excess of those used in design. Evidence of high horizontal acceleration is indicated by the shear cracks that developed in the solid walls.

Failures also occurred in the horizontal construction joints. Other factors, in addition to high horizontal acceleration, undoubtedly contributed to these failures. It normally would be expected that any joint movement at the first-floor level would be at the bottom, rather than at the top, of the floor construction as larger shear forces would be present in the lower level. Several factors that affected the efficiency of the joint should be listed. As the first floor was finished, the concrete was finished by troweling over the construction joint, at least to the reinforcing steel, making a smooth surface over a portion of the joint. Next, the pour for the wall above the floor was made in a deep form. This made it more difficult to clean completely the joint at the bottom, thus leaving foreign matter in the joint. Each of these reduced the bond across the joint and the joint's capacity to transmit shear. Additionally, little is known of the effectiveness of a joint that consists of concrete with stone aggregate on one side and lightweight aggregate on the other. Consideration, also, should be given to vertical acceleration when reviewing the joint failure, as the tension forces may have reduced the ability of the wall to resist the shearing force.

The horizontal fractures at the mezzanine level occurred in only two walls. These were the only 36-

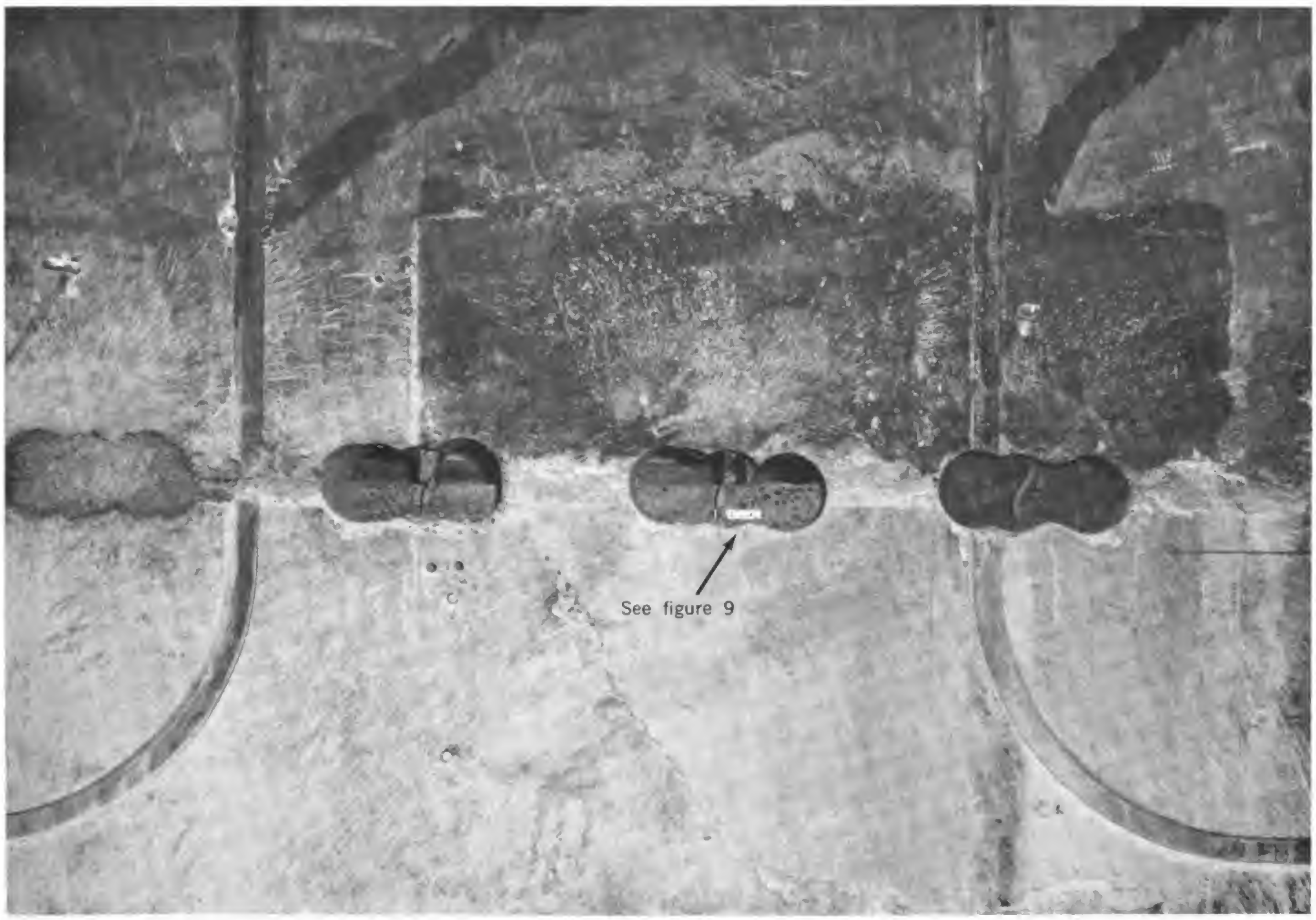

Figure 10.—Museum for Antique Cars. Typical keys cut in walls at fractured horizontal construction joints.

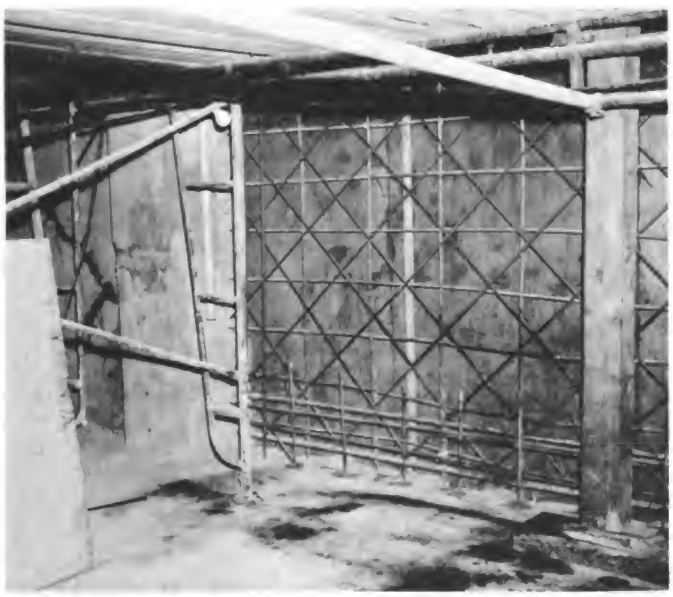

Figure 11.—Museum for Antique Cars. Reinforcing in place in wall panel to be replaced. Typical of north and south wall.

foot-high unsupported walls. It is possible that the flexibility of the walls perpendicular to their surface may have contributed to this failure.

RECOMMENDATIONS

The following items are recommended for further study based on the review of building damage:

1 Design values for shear in horizontal construction joints in shear walls should be developed for lightweight concrete used in conjunction with stone concrete, and for all stone concrete construction.

2 Methods of improved inspection of horizontal construction joints in concrete construction should be studied.

3 Code forces should be reviewed.

4 Design requirements considering vertical acceleration should be studied.

Goodwill Industries (15)

1132 Pico Street, San Fernando

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JAMES H. THOMPSON

Wilson & Thompson, Structural Engineers
Los Angeles, Calif.

GENERAL DESCRIPTION

This building, which was demolished following the earthquake, was located on a level site. Foundations were spread footings designed for a maximum bearing pressure of 2,500 psf.

The building was designed and built in 1948. The building code used for design criteria was the Uniform Building Code, 1946 edition. In accordance with building code requirements, the seismic design was based on a force level of 13.3 percent of the dead load of building elements, utilizing a box system.

The site is about 1 mile southwest and ½ mile southeast, respectively, of the surface faults and ruptures of the Sylmar and San Fernando segments of the San Fernando fault zone. Ground motion is believed to have been high in this area. Many old buildings constructed of unreinforced masonry, prior to 1933, were heavily damaged in the main shopping center several blocks away. Ground rupturing was not observed in the immediate area. Horizontal seismic force levels were estimated to be from 20 to 40 percent of gravity acceleration in this area.

The one-story building was about 50 by 100 feet in size and constructed of precast reinforced concrete elements (fig. 1). Walls were precast panels 4⅝ inches thick. Roof panels were precast concrete 5 inches thick, supported by concrete frames (three hinged arches) spaced 15 feet on centers that spanned 50 feet across the building (fig. 2).

The concrete frames resisted transverse seismic forces. The stresses at code force levels would have been well within the allowable range, and shear wall stress would have been less than 10 psi. No significant distress to the basic frames was observed. Longitudinal forces were resisted by the wall panels acting as shear walls.

The roof panels were connected by weld plates to the frames, but not to the wall panels or to each

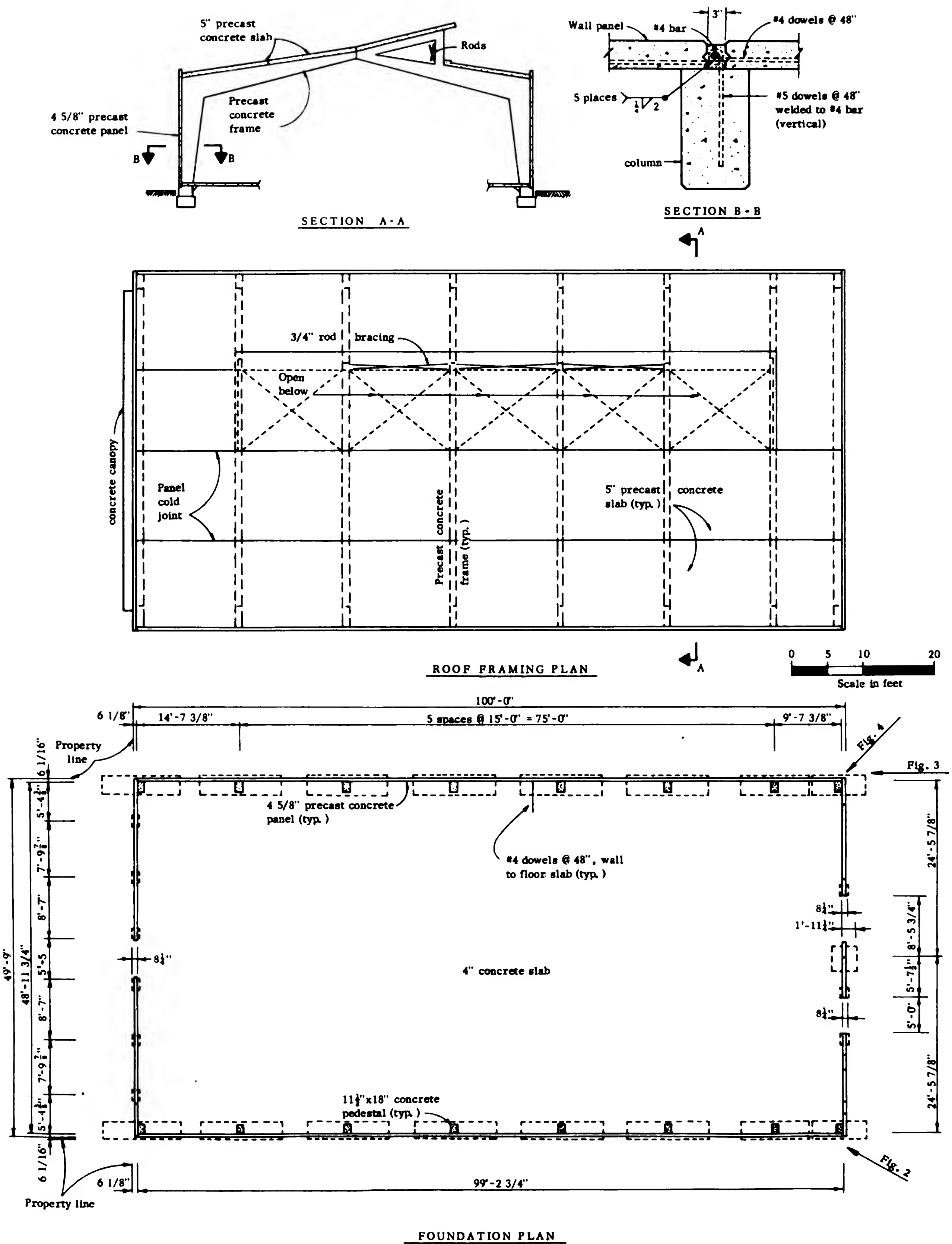

Figure 1.—Goodwill Industries. Roof framing plan and foundation plan.

Figure 2.—Goodwill Industries. Precast walls are braced for safety following earthquake. J. Thompson photograph.

Figure 3.—Goodwill Industries. Precast panel leaning at front corner. Rudy Gunnarson photograph.

Figure 4.—Goodwill Industries. Northeast corner. Note short dowels from concrete wall. D. F. Moran photograph.

other. The wall panels were connected to the frames by dowels hooked into the panel joint, and to each other by welding of No. 4 dowels at 4 feet on centers in the 3-inch-wide panel joint (fig. 1, section B–B). The wall panels also were connected to the floor slab with No. 4 dowels at a 4-foot spacing. Generally, the precast elements were tied together for continuity at relatively few locations. Total building continuity was less than is usually provided in modern buildings. There was no continuous chord member at the intersection of the roof plane with the plane of the walls.

EARTHQUAKE DAMAGE

Damage to the building occurred almost exclusively in the joints. Joints separated between adjacent wall panels. Dowels that were welded together in the joints pulled apart in most cases. Welds were of poor quality and generally were not of the specified size or length.

The wall panel separations at the joints allowed the panel to shift away from the building (figs. 3 and 4). Some panels also moved longitudinally and displaced adjacent panels, especially at the corners. Joints between the adjacent frames and roof panels were fractured or deformed in some areas. The floor slab was cracked, and dowels between wall panels and the floor slab had elongated or separated where the panels shifted away from the building.

Generally, precast elements were undamaged as individual pieces. Significant structural damage to the component parts was not observed. A lean-to addition at the rear of the building caused some local damage at the interface due to differential movement and hammering.

It is estimated that the replacement cost of the structure, prior to the earthquake, would be about $25 per square foot, or a total cost of $125,000. Repair to the structure could have been accomplished by removing and replacing the wall panels, repairing and reinforcing joints and continuity ties between elements, and repairing architectural items such as roofing, doors, finishes, and the like. It was estimated that these repairs would cost about $40,000. The owner of the building elected to demolish the structure and replace it with a modern design and layout that would be more suitable to present functional requirements.

CONCLUSIONS

The obvious cause of the damage to the building was the lack of continuity in the construction. Main points of weakness that contributed to the distress seem to be a lack of longitudinal continuity, both at the roof-wall interface and between adjacent panels. Anchorage of panels to the supporting frame and floor appears to have been inadequate to resist the seismic forces induced perpendicular to the wall.

RECOMMENDATIONS

Buildings of this type could be made more resistant to seismic forces by providing continuity reinforcing throughout the building to force the various elements to act together as a monolithic structure. The optimum condition of continuity would be equivalent to that obtained in a monolithically cast-in-place structure with properly designed and placed reinforcing steel. That optimum may never be reached, but substantial improvement over many current practices should be sought.

Detention Facilities

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Detention facilities in the heavily shaken area of the San Fernando earthquake suffered varying degrees of damage and disruption. The San Fernando Valley Juvenile Hall (16) facility was damaged quite heavily and one structure collapsed. There were no fatalities, however. Several inmates had to be extricated from their rooms by cutting and breaking security screens and windows because of blockage or damage at normal exit routes. A relatively large number of juveniles fled the establishment illegally. Camp Karl Holton Juvenile Facilities (17) sustained moderate structural damage to two buildings and a portion of the perimeter security wall.

Detention facility buildings are special when compared to the usual commercial or industrial buildings. These facilities are owned, operated, and maintained by a governmental agency; their design is usually based on criteria and specifications developed by the owner. It can differ from the normal building code requirements that govern the design of most other types of structures. Detention facility construction usually receives more supervision than normal building construction because of the security aspect.

Generally, building codes have not discriminated according to occupancy in their recommendations for earthquake-resistant design criteria. Therefore, buildings usually have not had a significantly different seismic design because of their occupancy. It has been suggested, most recently by structural engineers reviewing their own seismic code, that special design consideration perhaps should be given to occupancy relative to: (1) the importance of the continuous operation of the facility; (2) the magnitude of the potential disaster that building failure could precipitate; and (3) the special category of "forced occupancy," which includes the facilities described here.

Prisons, jails, and detention centers of various types are occupied by people who are incarcerated against their will. Since they are forced into occupancy of these facilities, the government is obligated

to provide them with a habitat that is reasonably safe relative to earthquake forces.

Governmental owners of detention facilities may have additional reasons, other than the forced occupancy, for a lower life hazard. The lower life hazard generally results in less property damage. Savings in reduced earthquake damage repair costs may offset extra construction costs over a long period of time. Another consideration would be increased prisoner security.

Specific conclusions and recommendations are given at the end of Building Reports 16 and 17.

San Fernando Valley Juvenile Hall (16)

15900 Filbert Street, Sylmar

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Photographs provided by James H. Thompson unless otherwise noted.

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GENERAL DESCRIPTION

The building site lies in the northwest end of the San Fernando Valley, just north of San Fernando Road between Filbert and Yarnell Streets. The site area is approximately 30 acres, rectangular, 1,041 by 1,382 feet, with an irregular northeast corner (fig. 1). The original grades sloped down about 60 feet from the northwest corner to the southeast. Soil conditions beneath the site are extremely variable, from a hard dense conglomerate at the west portion of the site to moderately soft silt and silty sand beneath the lower portion of the site, as shown on boring logs (fig. 1). Cuts up to 15 feet were required on the higher portions, and fills of about 7 feet maximum were required at lower elevations. The foundation report recommended spread footings underlain by a minimum of 2 feet of compacted fill, with a design soil pressure of 2,500 psf. A one-third increase in pressures was allowed for wind or seismic loads.

There are 12 structurally separated buildings on the site (fig. 1). The wall construction is reinforced hollow concrete block bearing and shear walls with concrete floor slabs on grade and concrete roofs. Structurally supported floors and roofs are either pan-joist systems or one-way slab systems, framing to walls or concrete beams and columns.

The building code used for design criteria was the Los Angeles County Building Laws, 1958 edition, which was based on the Uniform Building Code, 1958 edition. The project was built between 1962 and 1965. Seismic design was based on a force level of 13.3 percent of the dead load of building elements, utilizing a box system, and was generally in accordance with building code requirements.

There were severe permanent ground movements at this site, and these contributed to the severe structural damage.

Following is a general description of all 12 buildings and the damage they sustained. A more detailed

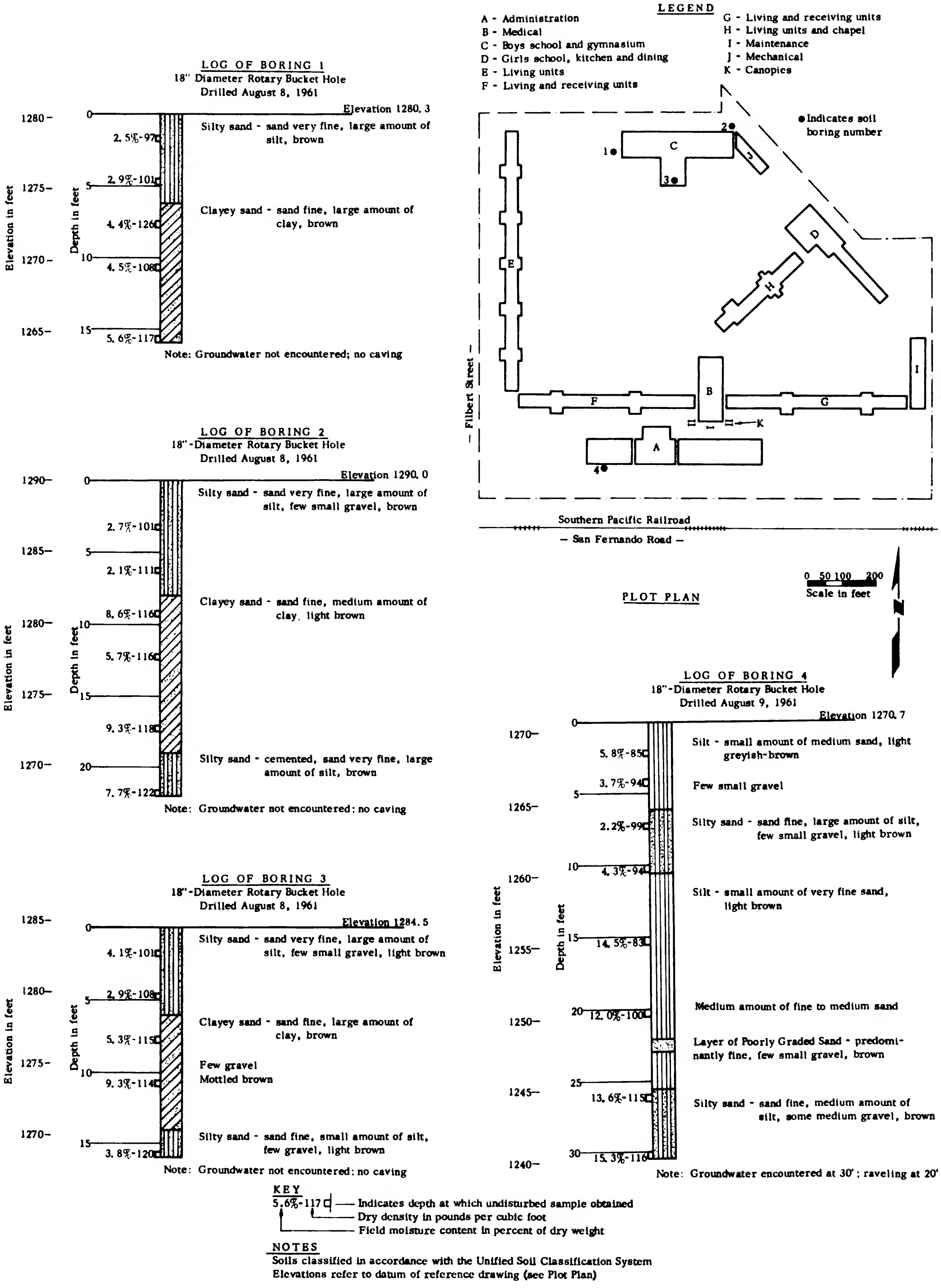


Figure 1.—San Fernando Valley Juvenile Hall. Plot plan and log of borings.

description is presented for the administration building west wing and the boys' school and gymnasium.

ADMINISTRATION BUILDING

West Wing—The one-story building is 70 by 136 feet with concrete columns, concrete beams, and block bearing and shear walls, and a concrete pan-joist roof system. Approximately 75 percent of the roof structure collapsed. The block walls were damaged heavily and those under the collapsed roof sections also collapsed. A more detailed analysis follows the security fence section.

Central Wing—The first story is 70 by 112 feet; the second story is 70 by 80 feet. Construction is concrete columns, concrete beams, and block bearing and shear walls, with a concrete pan-joist second floor and roof system. The floor slabs on grade sustained cracks several inches wide and vertical differential displacements of up to 1 foot. Heavy cracking occurred in the walls in the first story where the ground and floor slab cracking occurred. The second-story walls had some cracking. The second story had a permanent lateral displacement to the east of several inches.

East Wing—The one-story building is 70 by 243 feet. Construction is concrete columns, concrete beams, and block bearing and shear walls, with a pan-joist roof system. Heavy cracking occurred at grade and in the walls, similar to that at the central wing.

The three wings of the administration building are independent structures with a 2-inch separation joint between each of the units.

MEDICAL BUILDING

The one-story building is 73 by 184 feet with a partial basement (approximately 30 by 104 ft), block bearing and shear walls, and one-way slabs at the first floor (over basement) and roof. Pan-joist construction is used for the roof overhang only, around the north half (approximately) of the building. There is a 2-inch structural separation at the roofline on both sides of the adjacent receiving buildings (east and west).

Generally, damage was relatively light when compared to other buildings. There was some cracking in the walls and the floor slabs on grade.

BOYS' SCHOOL AND GYMNASIUM

The main classroom wing is 75 by 325 feet. The gym portion of the structure, 67 by 80 feet, projects south of the classroom wing. The classroom wing has block bearing and shear walls, with a pan-joist roof system and a one-way slab over the corridor (10-ft span). The gym has a combination of block and concrete bearing and shear walls, steel trusses (64-ft span) and steel deck roof system, and two mechanical mezzanines (north and south) approximately 16 by 64 feet, with pan-joist floor and roof systems. The upper portion of the gym, for the depth of the trusses (6 ft), is a clerestory (continuous windows) with a diagonal steel rod wall bracing system between the roof level and the top of the masonry walls. The building is structurally separated by a 2-inch expansion joint from the mechanical building.

The east end of the classroom wing had relatively little damage. Some light cracking occurred in the walls and slabs on grade. The west wing is about the same, except in the area of the west-end wall where heavy damage occurred in the floor, walls, and roof structure. The gymnasium has some light wall cracks. The diagonal bracing in portions of the clerestory was damaged. A more detailed analysis follows the security fence section.

GIRLS' SCHOOL AND KITCHEN, DINING, AND GYMNASIUM BUILDING

The one-story classroom wing (east side) is 32 by 192 feet, with block bearing and shear walls and a pan-joist roof system. The kitchen and dining wing (west side) is irregular in shape and size, approximately 96 by 149 feet. About half of the first-floor area is a raised structural floor with concrete beams and a one-way slab. The walls are concrete (below raised first floor) and block bearing and shear walls. The main roof is of pan-joist construction. A partial second floor is 17 by 76 feet and has a pan-joist floor and roof system.

The kitchen, dining, and gymnasium portions of the structure appear to have little or no significant structural damage, except at the interface with the classroom wing. A separation joint was not provided and extremely heavy damage occurred. The classroom wing separated about 12 to 18 inches, and massive damage occurred to the walls, roof structure,

and floor slabs. The damage generally diminished toward the east end of the building where cracks in walls were approximately 1 inch wide.

LIVING UNITS BUILDINGS

The building is 734 feet long, composed of six living units about 95 feet long and 32 feet wide, and separated by five utility-lobby room units about 33 feet wide (parallel to the long dimension of the building) and 59 feet long. The 11 units of the building have different roof and floor elevations, stepping down from north to south. The living units are two stories high, with block bearing and shear walls and one-way slab second-floor and roof systems. The floor and roof projection, beyond the face of the exterior walls (6 ft), are pan joists. The utility-lobby room units have block bearing and shear walls (common with living units) and pan-joist roof systems. A partial second floor (mechanical) is framed with one-way slabs on block walls. There are no structural separations between units of this building, although roof and floor levels are offset at various points.

Block walls were damaged heavily throughout the building. Many walls were shattered and buckled to some degree. The most severe damage occurred at points of floor and roof diaphragm discontinuity. Hammering damage was evident in many cases.

LIVING UNITS AND WEST RECEIVING UNIT BUILDING

The building is 516 feet long, composed of living units 96 and 192 feet long, both 32 feet wide, and a receiving unit 163 feet long and 32 feet wide. Two utility-lobby room units, 33 feet wide and 59 feet long, separate the living units. The utility-lobby room units have higher roof levels than the living and receiving units. The living units are two stories high and have block bearing and shear walls and one-way slab second-floor and roof systems. The floor and roof projection, beyond the face of the exterior walls (6 ft), are pan joists. The utility-lobby room units have block bearing and shear walls (common with living units) and pan-joist roof systems. A partial second floor (mechanical) is framed with one-way slabs on block walls. There are no structural separations between units of this building, although roof and floor levels are offset in some areas.

Damage is generally of the same type and magnitude as previously described in the living units building.

LIVING UNITS AND EAST RECEIVING UNIT BUILDING

The building is 537 feet long, composed of living units 100 and 199 feet long, both 32 feet wide, and a receiving unit 163 feet long and 32 feet wide. Two utility-lobby room units, 33 feet wide and 59 feet long, separate the living units. The utility-lobby units have higher roof levels than the living and receiving units. The living units are two stories high, with block bearing and shear walls and one-way slab second-floor and roof systems. The floor and roof projection, beyond the face of the exterior walls (6 ft), are pan joists. The utility-lobby room units have block bearing and shear walls (common with living units) and pan-joist roof systems. A partial second floor (mechanical) is framed with one-way slabs on block walls. There are no structural separations between units of this building, although roof and floor levels are offset in some areas.

Damage to the west part of the structure was relatively light, with moderate cracks in walls and floor slabs. The east portion of the building had several areas of extremely heavy damage to floors, walls, and the roof structure. Damage at the separation joint on the east end was very severe.

LIVING UNITS AND CHAPEL BUILDING

The living units are 240 feet long and are connected at the south end to the chapel building for a total length of about 308 feet. The living units are one story high, with block bearing and shear walls and one-way slab roof systems. The roof projection, beyond the face of the exterior walls (6 ft), is pan joists. The utility-lobby room is about 52 by 49 feet, with a pan-joist and concrete beam roof system and some block walls extending to the roof. The chapel is irregular in shape, about 68 by 61 feet, with three different roof levels, block bearing and shear walls, and several concrete walls. The roof system is composed of pan joists and concrete beams, with some walls extending up to the roof levels.

Damage to the northeast living units portion of the building was relatively light, with some moderate cracks in walls and floor slabs. The utility-lobby

room area was damaged very heavily. Ground cracks up to about 24 inches in width and having a differential vertical displacement of up to 12 inches extended through this area. A ground fissure up to 24 inches wide and 6 feet deep occurred along the west side of the southwest living units area. Walls and framing adjacent to the fissure were damaged heavily. The chapel building had light-to-moderate wall damage, except in areas where ground cracking occurred.

MAINTENANCE BUILDING

The one-story building is 204 by 49 feet with block bearing and shear walls, in combination with several concrete columns and roof beams. The roof is constructed with a pan-joist system. The building is structurally separated by a 2-inch expansion joint from the building on the east side.

Relatively light damage—some light floor and wall cracks—occurred at the south portion of the building. Local distress at the separation joint with the building to the west was evident. Heavier cracking, up to 1 inch, of the floor and the wall occurred at the north end of the building.

MECHANICAL BUILDING

The building is one story, 145 by 32 feet (irregular in shape, with a 114-ft west wall), and supported by block bearing and shear walls. Concrete pan joists form the roof system. The building is structurally separated by a 2-inch expansion joint from the building at the northwest end.

Most of the walls had moderate-to-heavy cracking. Interior transverse walls were damaged heavily. The southeast end wall was shattered completely. Some moderate damage occurred at the separation joint at the northwest end.

CONCRETE CANOPIES

The canopies are constructed utilizing a concrete pan-joist roof system, 16 by 26 feet, spanning between two concrete girders. The girders are supported on single concrete columns with 9-foot-long by 1½-foot-wide footings. There are five canopies located around and between the medical building and the administration building (fig. 7).

No significant structural damage was observed. Some very light cracking at the column-girder joint was noted. At several points where the steel security fence attaches to the concrete, the connection ruptured and pulled out chunks of concrete.

SECURITY FENCE

The fence was constructed with reinforced concrete blocks, 8 inches thick, with intermittently spaced 16- by 16-inch block pilasters with four No. 9 vertical bars. The wall footing is a continuous 8-inch-deep by 12-inch-wide pad. The pilaster footing is a 3- by 6-foot pad. Some fences do not have pilasters, in which case the vertical bars are No. 4 at 24 inches and the footing is 3 feet wide. Other details are similar to the typical 8-inch block fence. The walls are reinforced with No. 3 bars at 24 inches on center (vertical) and horizontal wall mesh at 16 inches on center. Two No. 4 horizontal bars are at the top of the wall. Only cells containing reinforcing were filled with grout. Most of the walls were about 15 feet in height.

Most walls received some damage; however, total collapse did not occur along the main wall lines. Many areas were leaning (up to several feet) and cracked.

DETAILED DISCUSSION OF ADMINISTRATION BUILDING WEST WING

The one-story building is 70 by 136 feet with concrete columns, concrete beams, block bearing and shear walls, and a pan-joist roof system. The roof framing is supported on concrete beams and columns, except for several interior bearing walls (fig. 2). Walls parallel to the roof joists are built up solid to the roof level to act as shear walls (fig. 3).

The 8-inch block walls are reinforced with No. 4 vertical bars at 24-inch spacing and wall mesh in horizontal mortar joints at 24-inch spacing. The wall mesh is a welded ladderlike reinforcement with 9-gauge longitudinal wires (at 6-in. spacing) and 9-gauge cross wires (at 16-in. spacing). Only those vertical block cells containing reinforcing were required to be filled. The 12-inch block walls are reinforced with the same reinforcing as the 8-inch wall, except that the wall mesh width is 10 inches. All block cells were required to be filled with grout in the 12-inch walls.

Approximately the west half of the roof structure collapsed, along with most of the north portion of

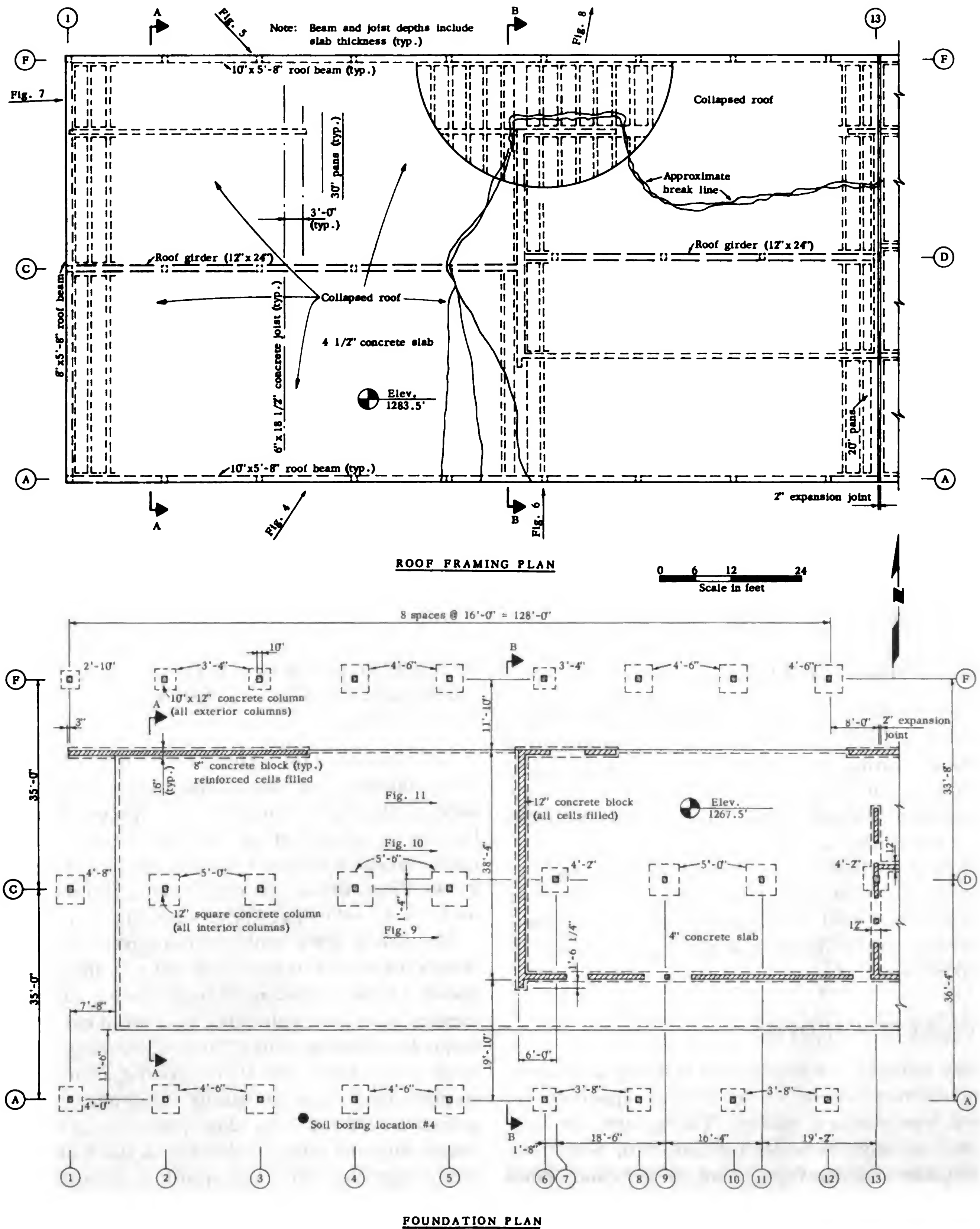


Figure 2.—San Fernando Valley Juvenile Hall. Administration Building, west wing. Roof framing plan and foundation plan.

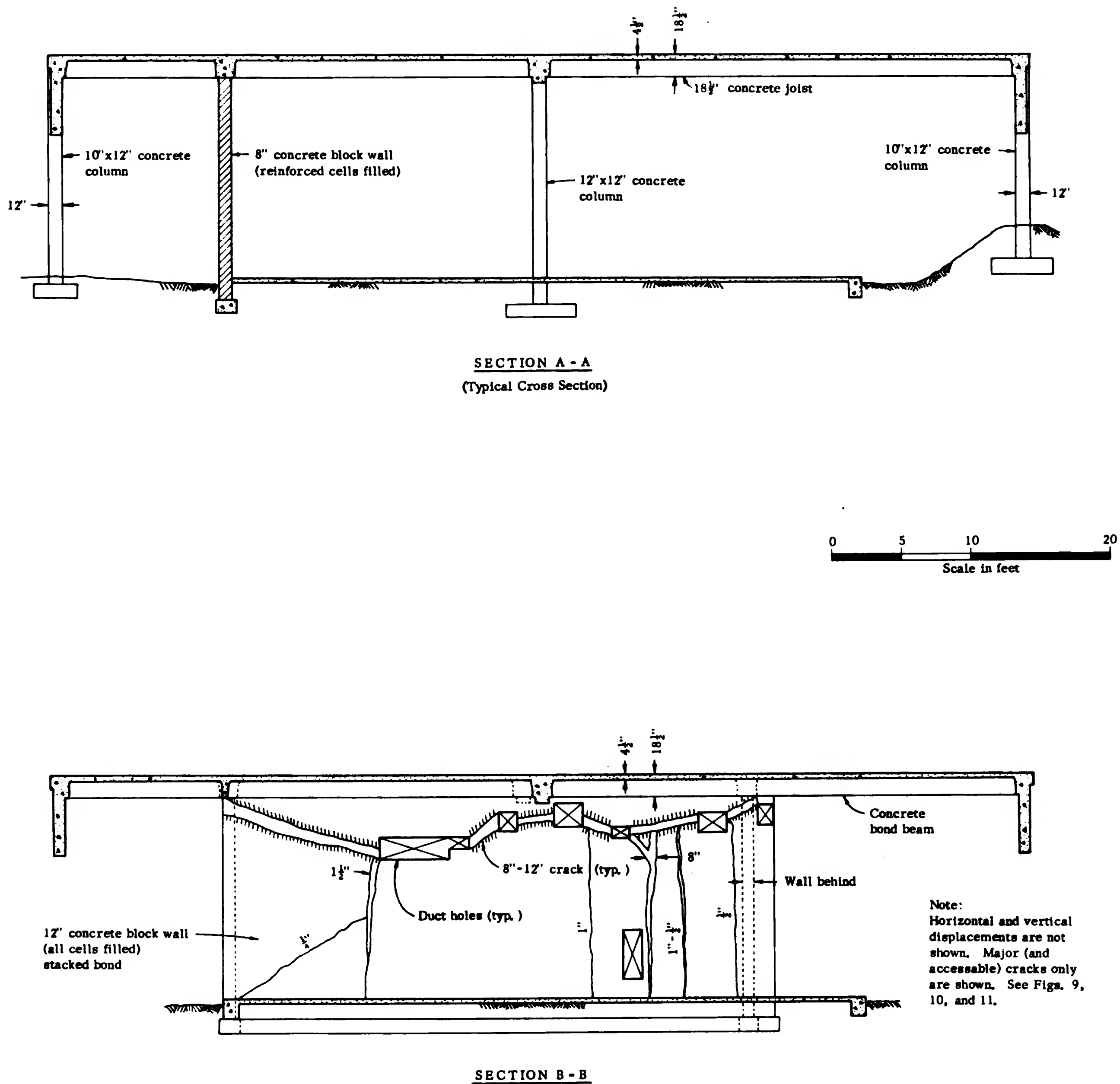


Figure 3.—San Fernando Valley Juvenile Hall. Administration Building, west wing. Cross section and wall elevation.

the northeast quadrant as indicated on the roof plan (fig. 2 and figs. 4 through 8). About 40 feet of block wall near the northwest corner and about 6 feet near the northeast corner collapsed under the falling roof. The principal north-south shear wall (12-in. block, stacked bond) was damaged very heavily, sustaining cracks from ¼ to about 12 inches wide (figs. 3, 9, 10, and 11). Massive cracking occurred between the north end of the 12-inch wall and the first duct hole near the top of the wall, and along the top of the wall between duct holes. The north portion of the wall leaned to the west several inches. The south portion of the wall tilted (in the plane of the wall) to the south about 8 inches. The abutting wall at that point also was leaning south (perpendicular to its wall plane). The remainder of that wall, between lines 9 and 13, was relatively undamaged. The short lengths of wall along line 13 also were undamaged, except the piers on each side of the concrete column on line D had light cracks.

The critical element in the seismic force-resisting system appears to have been the 12-inch block wall

Figure 4.—San Fernando Valley Juvenile Hall. West wing of Administration Building, looking northeast.

Figure 6.—San Fernando Valley Juvenile Hall. West wing of Administration Building. South wall line roof beam at main fracture line.

Figure 5.—San Fernando Valley Juvenile Hall. West wing of Administration Building, looking southeast.

Figure 7.—San Fernando Valley Juvenile Hall, looking east along north wall line of west wing of Administration Building. Note concrete canopies in background.

(fig. 3, section B–B). It is located near the center of mass of the building and would be required to resist nearly 100 percent of the north-south forces, plus some additional load due to rotation of the diaphragm. Based on the code requirements for a base shear of 13.3 percent of the dead load, the unit shearing stress in the wall would be about 37 psi. This assumes a gross section without taking the wall openings into account. A net section at a critical horizontal plane extending through the duct holes would produce a unit shear stress of about 56 psi. Ultimate shear stress for this type of construction has been established by tests (*Shear in Concrete Masonry Piers,* California State Polytechnic College, Pomona, Calif.) to be in a range of about 150 to 200 psi. The very heavy cracking (up to 12 in. wide) at this point and the evidence of horizontal movement (approximately 8 in.) of parts of the wall seem to indicate that the building shear wall system failures began along this wall line and probably precipitated other failures in the building. Special jamb reinforcing at openings was detailed on the drawings but could not be located when inspecting the wall.

Failures in the west portion of the building may have triggered the main roof system collapse, and could have resulted from excessive diaphragm translation and rotational displacement allowed by the shear wall failure. The tops of the columns were fixed rigidly by the 5-foot 8-inch-deep perimeter beams. Under excessive sideway displacements, a col-

Figure 8.—San Fernando Valley Juvenile Hall, looking from roof of west wing of Administration Building, north toward living and receiving units.

Figure 10.—San Fernando Valley Juvenile Hall. West wing of Administration Building. Massive cracking between duct holes in 12-inch block shear wall.

Figure 9.—San Fernando Valley Juvenile Hall. West wing of Administration Building, looking at south end of 12-inch block shear wall.

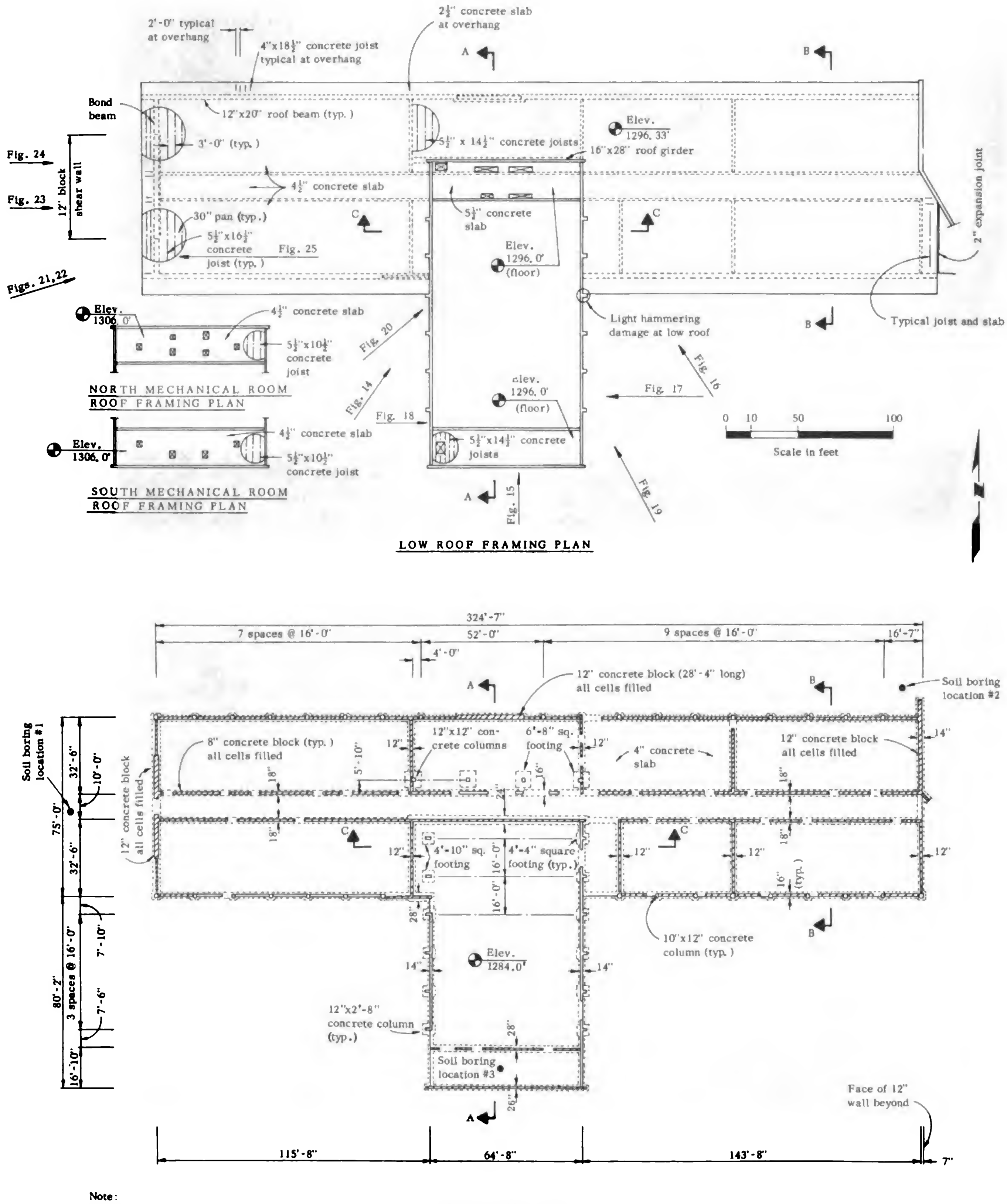
Figure 11.—San Fernando Valley Juvenile Hall. West wing of Administration Building. Massive cracking, near north end of 12-inch block shear wall, in line with duct openings.

umn failure seems likely at the column-beam joint. The east-west cracking and failure of the roof joists just north of and parallel to line D may have been contributed to by tension in the diaphragm as it tried to rotate about the 12-inch block wall. Excessive tension stress in the top reinforcing, which was spliced near this point, due to vertical acceleration forces, and vertical settlement probably contributed to the distress also.

Many possible sequences of failure can be postulated, but, in any case, the key factors to the actual collapse appear to have been the massive shear failure in the 12-inch block wall, the roof diaphragm rotation and translation, the fixity of the exterior columns by the deep perimeter girder, the building acceleration far in excess of the code assumptions, the possibility of significant vertical acceleration, and the occurrence of differential ground movement.

DETAILED DISCUSSION OF BOYS' SCHOOL AND GYMNASIUM

The main classroom wing is 75 by 325 feet. The gym portion of the structure projects south of the classroom wing 67 by 80 feet. The classroom wing has block bearing and shear walls with a pan-joist roof system and a one-way slab over the corridor (10-ft span). The gym has a combination of block and concrete bearing and shear walls, steel trusses (64-ft span), and a steel deck roof system. The two



Note:
Refer to Fig. 13
for Sections A-A,
B-B, and C-C

Figure 12.—San Fernando Valley Juvenile Hall. Boys' school and gymnasium. Low roof framing plan and foundation plan.

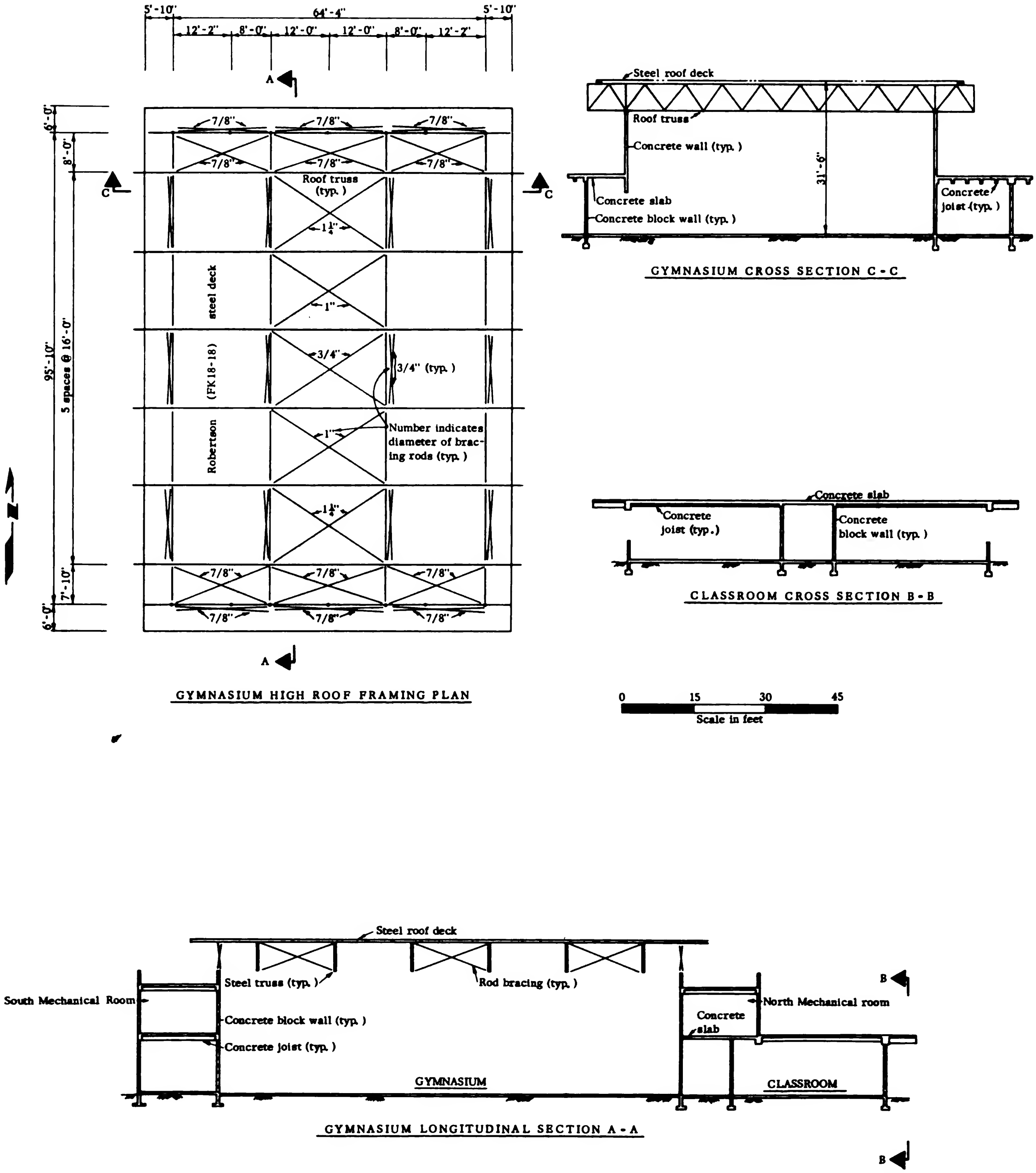


Figure 13.—San Fernando Valley Juvenile Hall. Boys' school and gymnasium. High roof framing plan, gymnasium cross section C–C, classroom cross section B–B, and gymnasium longitudinal section A–A.

mechanical mezzanines (north and south) are approximately 16 by 64 feet with pan-joist floor and roof systems. The upper portion of the gym, for the depth of the trusses (6 ft), is a clerestory (continuous windows) with a diagonal steel rod wall bracing system between the roof level and top of the masonry walls (figs. 12, 13, and 14). The building is separated structurally from the mechanical building by a 2-inch expansion joint.

The 8-inch block walls are reinforced with No. 4 bars at 16-inch spacing in each direction, and all cells are grouted solid. The 12-inch block walls are reinforced with No. 4 bars at 16-inch spacing in each direction on each face of the wall, and all cells are grouted solid. The 8-inch concrete walls at the gym are reinforced with No. 4 vertical bars at 16-inch spacing and No. 4 horizontal bars at 10-inch spacing.

This building was designed in accordance with the State building code for schools, California Administrative Code, Title 21, Public Works.

The east wing of the classroom buildings received no significant structural damage (figs. 15 and 16). The gymnasium portion was damaged in the upper framing where the steel roof deck seismic forces are transferred into the concrete walls. Diagonal steel rods in the east and west upper windows were damaged heavily (fig. 17). About half of the rods (7 or 8) were ruptured completely, usually at the threaded connection, and the remaining rods were stretched extensively. The upper window glass was broken out at the east and west walls. The connection of the diagonal rods at the top of the concrete wall was fractured where it is intersected by the bottom of the truss (figs. 18 and 19). Concrete was cracked and spalled on most of these connections along the east and west walls. The diagonal rods at the north and south wall clerestory were not broken or significantly stretched. One rod at the west end of the south wall was stretched slightly so that the sag was noticeable. No sagging was observed at the other rods. Windows were not broken at the north and south wall windows. The gym east wall (8-in. concrete) has diagonal cracks up to 1/8 inch in width on each side of the north door and a vertical crack, about 1/8 inch wide, over the south door (fig. 17). The gym west wall has larger cracks, up to about 1/4 inch wide, in the north bay between the first two exterior columns (figs. 18 and 20). Some vertical cracking also occurred in this bay. Generally, the gym portion of the building received very light damage, except at the

Figure 14.—San Fernando Valley Juvenile Hall. Boys' school and gymnasium, looking northeast at gym and west wing of school.

Figure 15.—San Fernando Valley Juvenile Hall. Boys' school and gymnasium, looking north at gym and two classroom wings. Note ground ruptures in foreground.

Figure 16.—San Fernando Valley Juvenile Hall. Boys' school and gymnasium, looking north at east wing of classroom building. No significant damage.

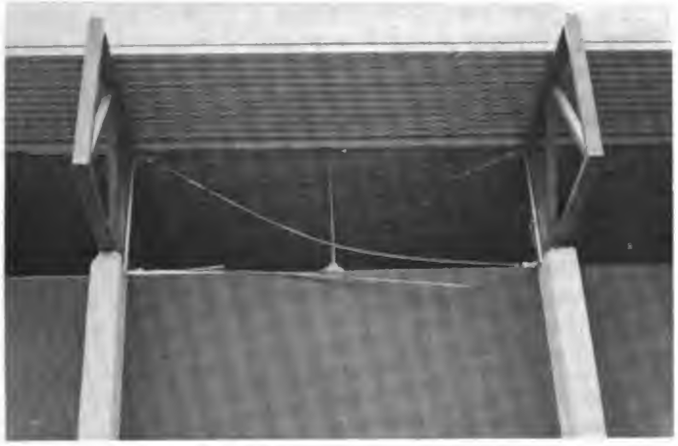


Figure 17.—San Fernando Valley Juvenile Hall. Boys' school and gymnasium, east wall of gym. Failed rod bracing.

Figure 18.—San Fernando Valley Juvenile Hall. Boys' school and gymnasium, west wall of gym. Failed rod bracing and cracks in concrete wall panel.

Figure 19.—San Fernando Valley Juvenile Hall. Boys' school and gymnasium, east wall of gym. Note cracks and spalling in concrete column.

east and west diagonal rods and the moderate wall cracking.

The west wing of the classroom building was damaged quite heavily at the west-end wall. The 12-inch block shear wall on both sides of the door sustained extensive fractures (fig. 21). The pier on the south side of the door was shattered, causing a large portion of the face shells of the blocks to spall off. Diagonal cracking up to about 1/2 inch wide occurred. The bottom corner of the wall pier (north side) was cracked heavily and was spalled and broken out in an area about 12 by 24 inches in size. Vertical jamb bars were exposed (fig. 22). The pier on the north side of the door was damaged; block face shells spalled off along the bottom course of the masonry units (fig. 23). The masonry wall spalled at the bottom corner at the south side of the pier, exposing jamb reinforcing (fig. 24). The interior 8-inch block wall also was ruptured at its juncture with the north pier. At the north end of the pier, an area of about 4 feet high and 5 feet wide was broken out completely (fig. 23). The 12- by 20-inch reinforced concrete bond beam over the door was completely ruptured. It sustained a diagonal crack 2 to 4 inches wide and local spalling. The parallel joists at the roof overhang also were fractured at this point with diagonal cracking (fig. 25).

The northernmost transverse 12-inch block wall, in line with the west wall of the gym, suffered some relatively moderate damage. Diagonal cracks up to about 1/4 inch wide occurred, and some spalling of block face shells was noted along the top of the wall and at some spots in the middle of the wall.

The damage at the west end shear wall appeared to be the type of failure where the walls are overloaded and fail in shear. The resulting pier translation and rotation grossly overstressed the relatively rigid, common connecting member (bond beam) across the tops of the wall. The rigid concrete bond beam completely fractured over the door because of its inability to sustain rotations cranked in from the piers at each side of the opening. The piers would have been stressed to about 14 psi at code force levels. Assuming a basic shear stress of 50 psi at the "first cracks," which seems reasonable for the type of material and pier configuration, the seismic force level would have been about 47 percent of gravity at the "first cracks." Obviously, the actual stresses were quite a bit higher than "first crack" intensity, although ultimate wall capacity probably was not

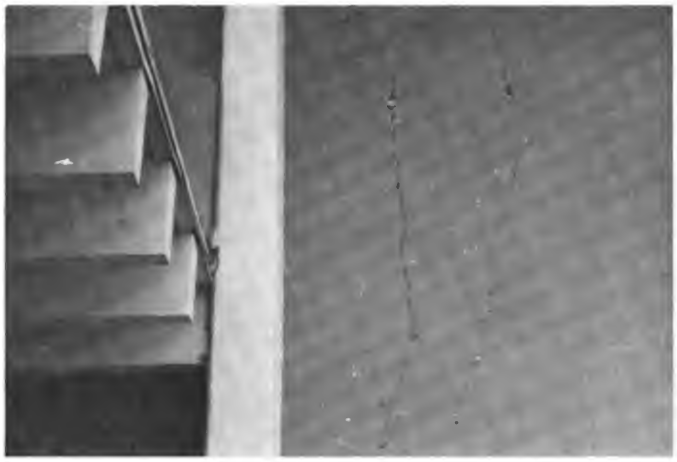


Figure 20.—San Fernando Valley Juvenile Hall. Boys' school and gymnasium. Wall cracks in west wall of gym. Note "hammering" effect on low roof.

Figure 23.—San Fernando Valley Juvenile Hall. Boys' school and gymnasium. West shear wall of boys' classroom. J. F. Meehan photograph.

Figure 21.—San Fernando Valley Juvenile Hall. Boys' school and gymnasium. West wall of west classroom wing.

Figure 24.—San Fernando Valley Juvenile Hall. Boys' school and gymnasium. West wing shear wall. J. F. Meehan photograph.

Figure 22.—San Fernando Valley Juvenile Hall. Boys' school and gymnasium. West wall of west classroom wing.

Figure 25.—San Fernando Valley Juvenile Hall. Boys' school and gymnasium. Concrete bond beam over door at west wall of west wing.

reached. Equating the actual cracks to a stress of about 150 psi, the seismic force level would have been about 141 percent of gravity.

The south wall of the west classroom wing had several very light hairline diagonal cracks in the first bay adjacent to the gym. The tops of the columns were cracked slightly, as evidenced by paint spalling, at their interface with the bottom of the concrete bond beam. Some flexural frame action apparently occurred in conjunction with the shear wall action along this line.

The west-end wall and the diagonal rods at the gym roof represent the "weak link" in the seismic-resisting system. Their failure indicates a gross seismic overload many times the code-prescribed forces.

SEISMIC FORCE LEVELS

The estimates of seismic force levels that acted on the buildings vary. Estimates based on analysis of yielded materials in building elements indicate a ground force level (acceleration) of from 30 to 70 percent of gravity. Wide variances in actual building responses seem apparent. They were caused by differences in dynamic characteristics and response of buildings and building elements, varying soil conditions, and location and effect of ground ruptures throughout the site.

GROUND RUPTURES

The building site lies at the head of a landslide area called the Juvenile Hall slide. The western boundary of the slide bisects (approximately) the site from the southwest corner to the northeast corner. The eastern boundary of the slide area extends through the southeast corner of the site. The ground surface ruptured at many points along these main slide lines. Vertical, horizontal (parallel), and horizontal (perpendicular) displacements and associated ground failures occurred in these areas. Generally, where these differential ground movements occurred at buildings, heavy damage resulted (figs. 26 and 27). Where the slide boundary crossed the Southern Pacific Railroad, just south of the site (fig. 1), track displacements (horizontal) of about 4 feet were noted. Massive freeway bridge damage occurred within a half mile of the Juvenile Hall site.

Undoubtedly, a significant portion of the damage to buildings resulted from ground rupture and/or a combination of ground rupture and extremely high accelerations, both horizontally and vertically.

For additional information on ground movements, see "Ground Displacement at San Fernando Valley Juvenile Hall During San Fernando Earthquake," in Volume III.

COST ESTIMATE

The administration building west wing will have little salvage value. Some of the mechanical equipment, such as heating and air-conditioning, could be saved. This savings probably will not be equal to the cost of demolition of the existing building. Assuming this salvage savings equals demolition costs, the replacement cost of the building is equal to its value prior to the earthquake. This could be estimated at about $28 per square foot, or a total cost of about $270,000.

Figure 26.—San Fernando Valley Juvenile Hall. Ground rupture at building.

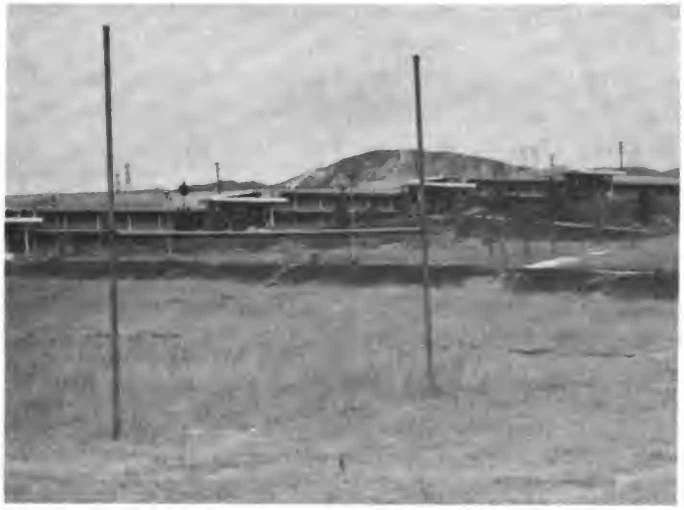


Figure 27.—San Fernando Valley Juvenile Hall. Ground rupture in foreground. Living units E in background.

The boys' school and gymnasium building had an estimated replacement cost of about $700,000, not counting demolition. The estimated repair cost was about $165,000.

CONCLUSIONS

Damage to the various structures generally was quite heavy relative to what normally would be expected from the magnitude of the earthquake, the building code assumed levels of safety, and the type of construction.

The excessive damage can be explained, in part, by extensive ground faulting, unusually high ground accelerations, and general earthquake-effect amplifications associated with the Juvenile Hall slide phenomenon. Significant vertical acceleration, in conjunction with the horizontal acceleration forces, probably contributed to the damage.

Several observations were noted in the construction details that may be helpful in improving the performance level of this type of construction. Concrete block walls with all cells grouted and horizontal reinforcing steel seemed to perform better than walls with horizontal mesh reinforcing and vertical cells grouted at 24 inches. The 12-inch concrete block shear wall at the administration building west wing could have been strengthened along the top where the series of duct holes occurred. Structural separations of 2 inches were inadequate, in some cases, to prevent hammering. It was observed that architectural closures, caulking, and the like, compressed in the joints to reduce their effectiveness. Very long buildings without structural separation joints suffered heavy damage at points of diaphragm discontinuity (vertically). Low roofs attached to walls that extended up to support a high roof caused excessive damage at the interface point.

The linked shear walls at the west end of the boys' school structure represent a type of configuration that is difficult to rationalize. The concrete bond beam is usually too weak to act as part of a "frame" with the connected shear walls, and is not flexible enough to tolerate excessive end rotation. This problem usually is solved by reinforcing the beam as heavily as possible, with as much confinement (and resulting ductility) as geometry permits. Alternatively, joints sometimes can be built into the structure to allow end rotations.

RECOMMENDATIONS

Engineering design and construction practices could be revised in such a way that the resulting structures would be substantially more resistant to earthquake forces. The following suggested revisions would not make the resulting structure "earthquake proof," but they would increase the safety factor against earthquake damage.

1 Grout all cells solid in concrete block shear walls.

2 Provide more than the minimum amount of reinforcing steel, but in the form of bars, not wall mesh.

3 Prohibit the use of "stacked bond" pattern of coursing in shear walls.

4 Increase the width of expansion joints, and prohibit the inclusion of nonstructural material in the joints without taking into account the resulting reduction of effective joint width.

5 Provide extra strength and reinforcement at points of diaphragm and wall discontinuity in building elements.

6 Provide extra strength and design considerations for shear walls around wall openings.

7 Provide either adequate strength or adequate flexibility for linking members between shear walls.

8 Provide geological investigations for important facilities to determine potentially hazardous conditions.

Camp Karl Holton Juvenile Facilities (17)

16691 North Little Tujunga Canyon Road
Los Angeles County

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Photographs provided by James H. Thompson.

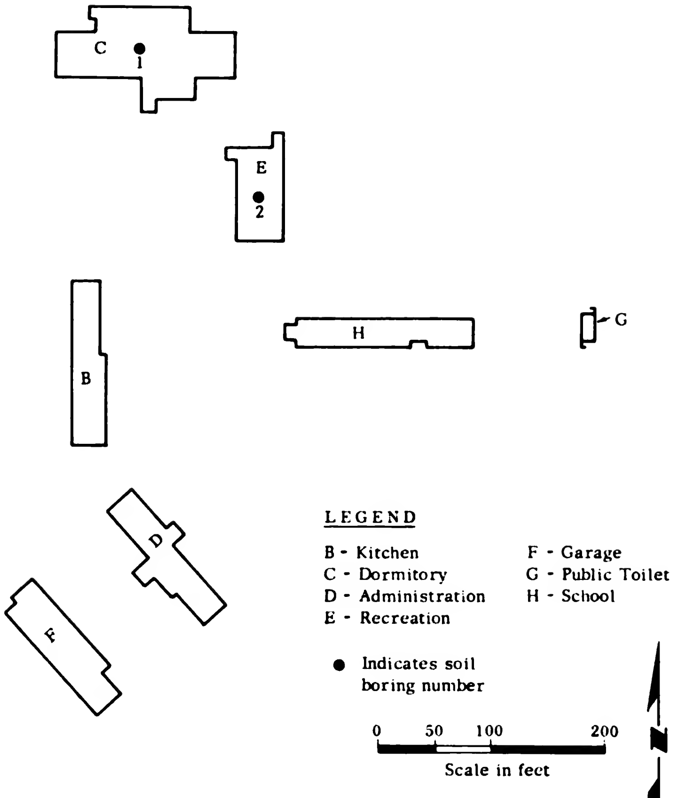
JAMES H. THOMPSON
Wilson & Thompson, Structural Engineers
Los Angeles, Calif.

GENERAL DESCRIPTION

The building site is in Little Tujunga Canyon in the foothills of the San Gabriel Mountains, about 2½ miles north of Hansen Dam and about 3 miles east of San Fernando. The building site is terraced and slopes down to the south and west with a total drop of about 45 feet. The soil conditions are generally coarse sand with variable amounts of gravel and cobbles. The material is known locally as decomposed granite or disintegrated granite (refer to boring logs, fig. 1). Some of the soils were well cemented and were very difficult to excavate. Several of the buildings are on compacted fill of the native material. The foundation design was based on a soil pressure of 2,000 psf. The school building had a limiting dead load bearing pressure of 1,400 psf. Considerable cut and fill were required to adapt the building sites.

There are seven separate buildings on the site (refer to site plan, fig. 1). Wall construction is concrete block and floors are concrete slabs on grade. Roofs are generally of wood construction, except one building has a poured gypsum roof deck. The building code used for design criteria was the Los Angeles County Building Laws, 1956 edition, which was based on the Uniform Building Code. The school building was designed in accordance with California Administrative Code, Title 21. The structures were designed for a seismic force of 13.3 percent of the dead load of the building, utilizing the box system. The project was designed in 1957 and was built prior to 1960.

A brief description of the buildings and the structural damage sustained will be covered for five buildings. A more detailed description and discussion will be presented for the remaining two buildings.



PLOT PLAN

Boring #1				
	γ	M	S	
				Buff Silty D. G.
1	116.8	4.3	2.7	
2	126.3	3.4	2.3	Buff sandy D. G. with scattered gravel

Boring #2				
	γ	M	S	
1	116.0	4.3		Buff sandy D. G.
2	117.7	3.4		Hard granite spots
3	117.7	3.1		

LEGEND

γ - Dry unit weight pounds per cubic foot
M - Moisture content, percent
S - Shear strength, kips per square foot
1 - Undisturbed sample number 1
D.G. - Decomposed granite

LOG OF BORINGS

Figure 1.—Camp Karl Holton Juvenile Facilities. Plot plan and log of borings.

SCHOOL CLASSROOM BUILDING

The building is one story, 27 by 186 feet, with concrete block bearing and shear walls and a wood frame roof. The roof is a pitched roof with rafters and ceiling joist ties at 24-inch spacing, with 1- by 6-inch tongue and groove V-joint diagonal sheathing. Maximum diaphragm span is about 54 feet. No significant structural damage occurred. Concrete walks outside and adjacent to the building were cracked badly and buckled in several areas. This building was designed in accordance with the State building code for schools. For discussion of this code, see the paper "Public School Buildings."

KITCHEN MESS HALL

This one-story building is irregular in shape, about 30 by 148 feet, with concrete block bearing and shear walls and a wood frame roof. The south half (approximately) has a trussed rafter system similar to the school classroom building, and the remainder has wood trusses supporting 4- by 6-inch purlins with 2- by 6-inch tongue and groove V-joint diagonal sheathing. No significant structural damage occurred.

ADMINISTRATION BUILDING

This one-story building is irregular in shape, about 30 by 132 feet, with concrete block bearing and shear walls and a wood frame roof. About two-thirds of the building has a trussed rafter system similar to that of the school classroom building, and the remainder has a wood beam and purlin system with 2- by 6-inch tongue and groove V-joint straight sheathing. No significant structural damage occurred.

GARAGE BUILDING

The garage building is one story, 38 by 127 feet, with concrete block bearing and shear walls and a wood truss and beam roof framing system, supporting 2- by 6-inch diagonal tongue and groove V-joint sheathing. No significant structural damage occurred.

TOILET BUILDING

This is a small one-story building, 11 by 37 feet, with concrete block bearing and shear walls and a wood rafter roof system with 1- by 6-inch tongue and groove V-joint diagonal sheathing. No significant structural damage occurred.

DETAILED DISCUSSION OF RECREATION BUILDING

The one-story recreation building is 41 by 82 feet, with concrete block bearing and shear walls (fig. 2). There is a 12- by 12-foot appendage at the northeast corner with a wood rafter roof framing system at a level lower than the main roof. There is a walkway canopy along the west side of the building with wood rafters supported at the block wall and on a 6- by 8-inch wood beam at the exterior side. The 6- by 8-inch beam is supported on steel pipe columns. The main area of the building is framed with steel wide-flange rigid frames, with bases assumed pinned. The roof has 6- by 12-inch purlins, on a 5-foot spacing, that span between frames and support the 2- by 6-inch diagonal sheathing (figs. 2, 3, 4, and 5). The walls are all 8-inch concrete block, except the south end wall which is 12-inch concrete block. The 8-inch walls are reinforced with No. 4 vertical bars at 24-inch spacing and horizontal bars at various spacings. The 12-inch wall has No. 4 vertical and horizontal bars at 24-inch spacing, each way at each face of the wall. The building was designed for a seismic force of 13.3 percent of the dead load, in accordance with the building code.

A differential horizontal displacement occurred along the floor slab construction joint at the line of the center steel frame. The south side of the floor (at the joint) moved to the east about 3 inches. This caused a 3-inch crack to develop in the slab, at and parallel to the west wall. The bottom of the east wall displaced to the east about 3 inches. The magnitude of the cracks and wall displacements diminishes to zero at the south wall. The east wall was cracked heavily at this point (fig. 6). The floor along the south side of the central east-west floor crack heaved up about 1½ inches, and the original construction joint opened up to about 1½ inches in width. The floor along the north side of the crack seemed to be fairly stable, with no observed horizontal displacement. There were several large floor cracks, varying in width from ⅛ to ½ inch, in the north portion of the building.

The east wall was cracked and separated at the main floor slab construction joint crack (fig. 6). There was another large crack, up to 1 inch in width, at the northeast corner below the corner of the high roof (fig. 7). There were five other smaller cracks in the south portion of the wall. The west wall had four cracks, one each side of each of the two south frames. The largest crack, on the south side of the south frame, varied in width from ⅛ to ½ inch.

Heavy local cracking and spalling occurred at the top of the walls at the southeast and southwest corners (fig. 8). A similar condition occurred at the northeast corner (fig. 7). Generally, the wall tops were shattered and broken through the wall thickness at these points. There was some evidence of differential lateral movement between the top of the south wall and the roof construction.

There were numerous small cracks, ⅛ inch and less, in the small rooms (low roof area) at the northwest and northeast corners of the main building.

The walkway canopy along the west wall was distorted in two directions, horizontal displacement at the bottom and vertical settlement (fig. 5). Ground movements were large along the west side of the building, with cracks, heaving, and lateral displacements of up to 3 or 4 inches (figs. 9 and 10). The ground movement caused the pipe column footings to move away from the building (west) several inches and to settle several inches. The column tops were held in place by the roof framing. The column rotation was resisted at the top by the connection to the 6- by 8-inch beam, which had pulled loose from the framing as it rotated (torsion) about its longitudinal axis.

There were two distinct categories of damage—the floor and wall cracking (vertical) and the canopy damage, and the damage to the wall tops at the three corners.

The first type of damage was caused by excessive differential ground movement, both horizontal and vertical. The subgrade under the south-central part of the building shifted east, upward, and south to a lesser degree, resulting in heavy damage to the slab and walls in that area. The earth movement along the west wall was to the west, pulling concrete walkway slabs and pipe column bases away from the building. The main building footings were not affected noticeably by this movement. The fact that the wall cracks were all vertical, or nearly so, indicates a type of failure other than in-plane shear failure. Some overall building movement to the south of the south portion of the building seems to have occurred. This would explain the vertical cracks in the walls.

The second type of damage is more complex. It seems to have been related to a combination of fac-

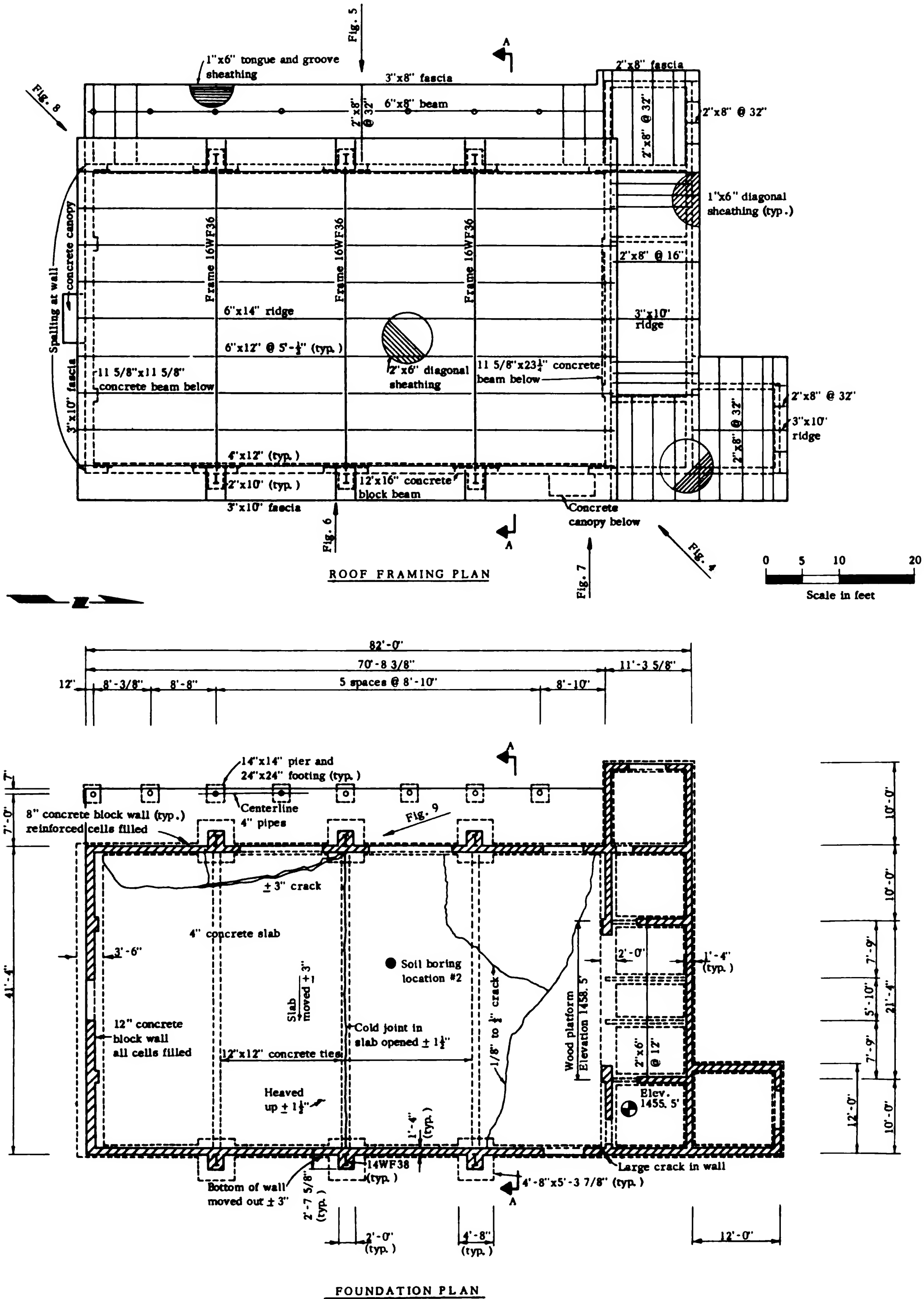
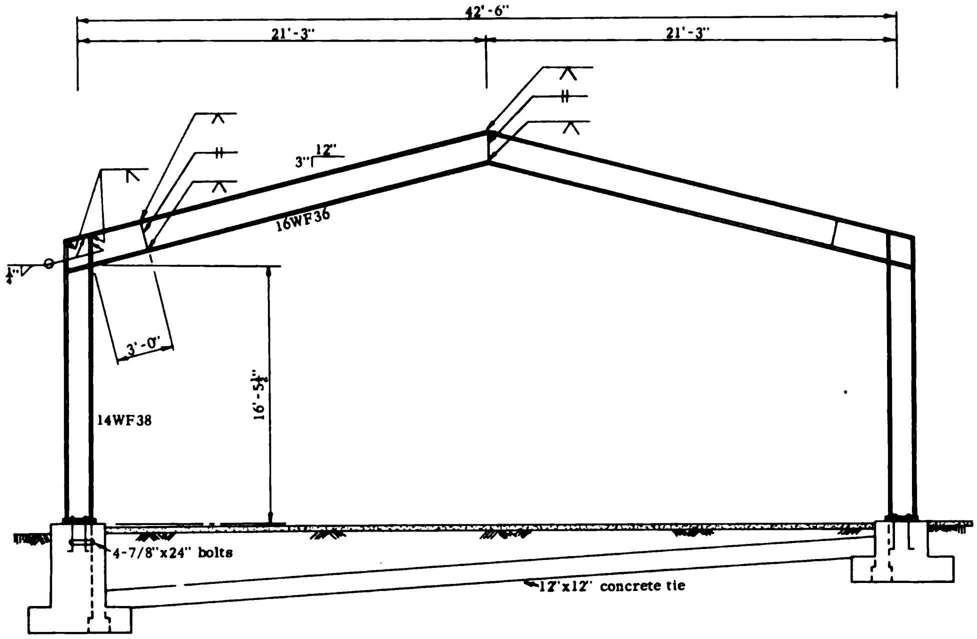
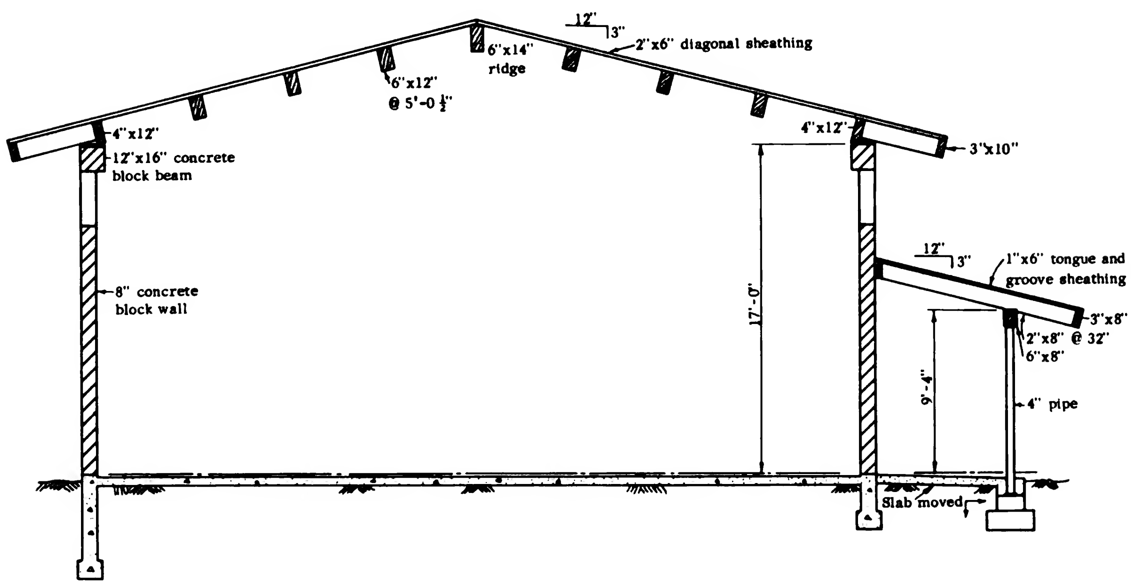


Figure 2.—Camp Karl Holton Juvenile Facilities. Recreation building E. Roof framing plan and foundation plan.



STEEL FRAME ELEVATION



SECTION A-A

Figure 3.—Camp Karl Holton Juvenile Facilities. Recreation building E. Steel frame elevation and section A–A.

Figure 4.—Camp Karl Holton Juvenile Facilities. Recreation building, looking southwest.

Figure 6.—Camp Karl Holton Juvenile Facilities. Recreation building. Large crack in east wall.

Figure 5.—Camp Karl Holton Juvenile Facilities. Recreation building, looking east.

Figure 7.—Camp Karl Holton Juvenile Facilities. Recreation building. Wall crack at northeast corner of high roof area.

tors including: the differential stiffness of the north- and south-end walls relative to the interior frames; the stiffness of the roof diaphragm relative to the walls and frames; and the diaphragm distortion characteristics. The roof diaphragm is considerably stiffer than the three interior steel frames. It would have a tendency to span the 70 feet between end walls, resisting the major portion of the east-west loads, until its capacity or the capacity of its connections to the shear walls (north and south) is exceeded and excessive deflections begin to occur. As the deflections increase, the amount of force resisted by the steel frames would increase. The end walls have virtually infinite rigidity relative to the steel frames. The relative displacement of individual members of a wood diaphragm, through nail bending, has been observed (Douglas Fir Plywood Association Laboratory, Report No. 55) to be a maximum at the corners. This type of displacement could have contributed to the corner distress as the diaphragm approached its yield capacity. Some bending (flexural) forces were probably set up in the corners of the walls as the diaphragm underwent flexural deflections. This would have resulted in a stress concentration at the top of the wall corners.

In summary, it seems likely that the roof diaphragm spanning between the end walls resisted the main portion of the east-west loads without significant support from the frames. The resulting shears overstressed the connection to the end walls, causing diaphragm distortion at the corners and inducing bending moments (in a horizontal plane) at the corner wall tops, which caused the cracking and displacement. At the northeast corner, the situation was

aggravated by a lower roof-wall intersection that appears to have caused a differential movement crack and subsequent hammering.

Another factor contributing to the excessive ground motion and earth failures is the strong possibility that all or part of the building was built on fill material. Exact confirmation could not be made, but the design drawings seem to support this concept.

Many possible combinations of building response could have contributed to the wall corner damage, but, in any case, it seems to be related to the relative stiffness of the resisting elements, the high seismic force levels, and the possibility of significant vertical accelerations. Building elements would not have been overstressed under code seismic force levels and design criteria.

Figure 8.—Camp Karl Holton Juvenile Facilities. Recreation building. Wall cracking at southwest corner. Note separation between canopy and wall.

DETAILED DISCUSSION OF DORMITORY BUILDING

The one-story building is irregular in size, with a 40- by 160-foot main building, a 22- by 90-foot attached north wing, and a 20- by 40-foot (approximate) attached south wing. Wall construction is concrete block bearing and shear walls, with a 2-inch poured gypsum roof deck. The roof framing system is primarily a wood beam framing system composed of wood beams and purlins (fig. 11). The north and south walls extend up in the center portion to support a high roof level 69 feet long (figs. 12 and 13).

The 8-inch block walls are reinforced with No. 4 vertical bars at 24-inch spacing and horizontal bars at various locations, usually near the roofline. The 12-inch walls are reinforced with No. 4 bars at 24-inch spacing for both horizontal and vertical bars on each face of the wall. Only reinforced cells were required to be filled. Concrete beam-column systems support the upper east and west walls of the high roof area and are tied into concrete frames located in the north wing (figs. 11 and 14). The concrete frames provide the lateral support to the building at the high roof area.

Ground movement and cracking were heavy at this location. Exterior sidewalks at the east end were cracked and dislocated (fig. 15). Ground fissures several inches wide occurred in the dirt driveway just

Figure 9.—Camp Karl Holton Juvenile Facilities. Recreation building. Sidewalk and ground rupture along west wall.

Figure 10.—Camp Karl Holton Juvenile Facilities. Recreation building. Sidewalk rupture at west wall.

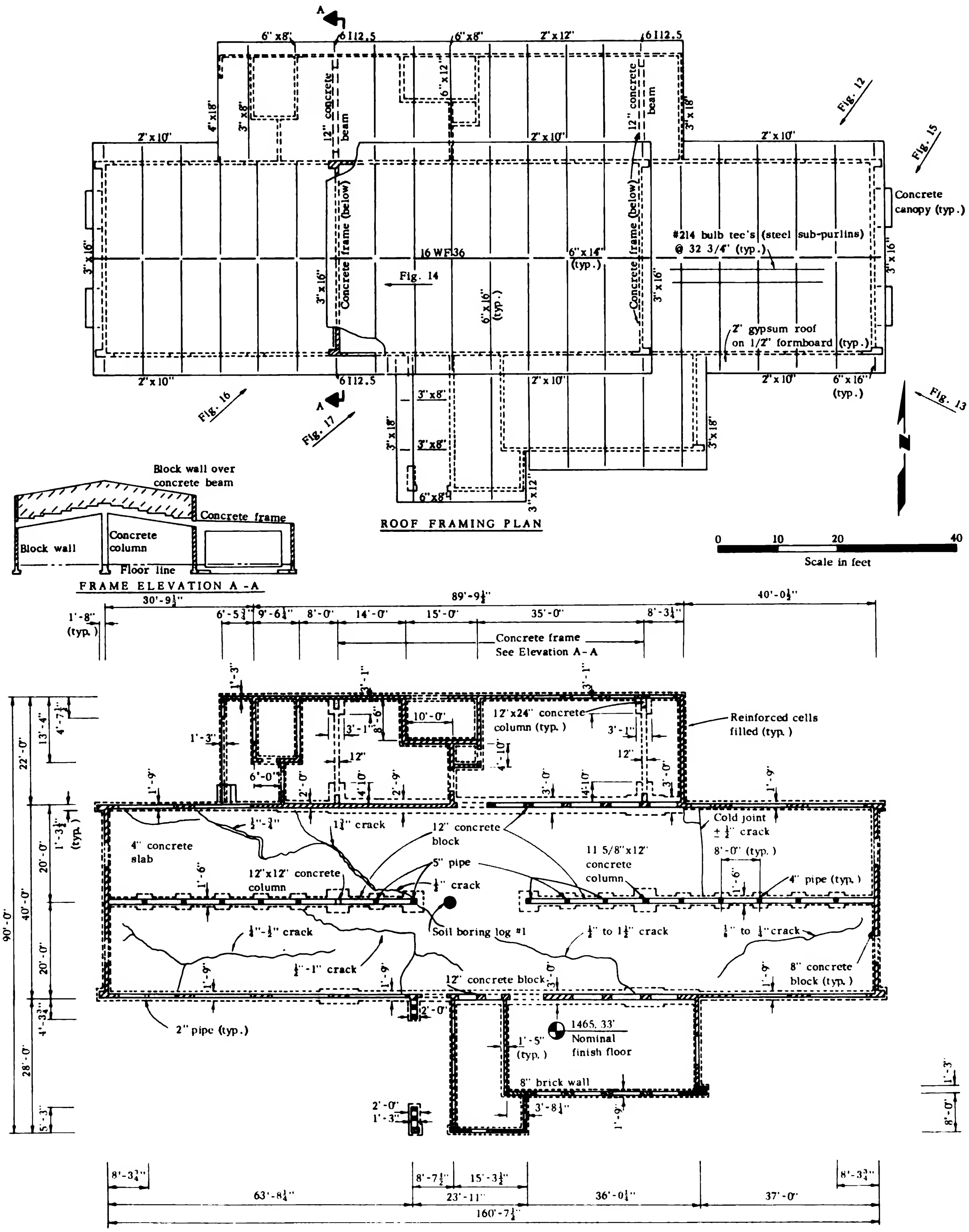


Figure 11.—Camp Karl Holton Juvenile Facilities. Dormitory building C. Roof framing plan and foundation plan.

Figure 12.—Camp Karl Holton Juvenile Facilities. Dormitory building, looking southwest.

Figure 13.—Camp Karl Holton Juvenile Facilities. Dormitory building, looking west.

Figure 14.—Camp Karl Holton Juvenile Facilities. Dormitory building, south wall, west end. Concrete beam supporting upper wall and high roof.

Figure 15.—Camp Karl Holton Juvenile Facilities. Dormitory building. Sidewalk ruptures at east end of building.

north of the building, and landslides occurred on the paved road to the west. Several large cracks ½ to 1½ inches wide extended through the building. The floor slab in the center southwest portion of the building appears to have shifted south several inches. The masonry pier, under the high roof wall line, in the south wall has been shifted out (south) at the bottom 3 to 4 inches (figs. 16 and 17). Seven piers along the south wall line are cracked, with diagonal shear cracks varying in width from hairline to ¾ inch. The three piers in the south end of the wall have the largest cracks. These piers are constructed of a 4-inch concrete block unit on the inside and brick masonry units laid up on the outside. The inner and outer layers of masonry separated on some piers. There are six small (hairline to ⅛-inch) cracks along the main building north wall, generally at spots where floor slab cracks terminate at the wall. No heavy, structurally dangerous cracking or displacement occurred as it did on the south wall. There are several very light (hairline) diagonal cracks in the upper (above the low roof level) wall areas. One crack at the upper southwest corner, in the pier directly over the cracked pier below (fig. 17), is about ¼ inch wide. No damage to the gypsum roof deck was observed.

A new addition was under construction at the south-side wall at the west end. Several partially completed block walls were damaged by the earthquake.

The damage in this building was of two types: damage caused by excessive differential ground

Figure 16.—Camp Karl Holton Juvenile Facilities. Dormitory building, south wall, west end. Note building material on ground from partially completed additions.

Figure 17.—Camp Karl Holton Juvenile Facilities. Dormitory building. Cracked wall piers in lower and upper piers, south wall.

movement and subgrade shifting, and that caused by the building shaking (acceleration) and the subsequent overstressing of resisting elements.

The heavy floor slab cracking and shifting of the subgrade appear to have been caused by differential ground movement and rupturing. Wall cracking, in some cases, also can be attributed to the ground rupturing and displacement.

The diagonal cracks in wall piers along the south wall were the type caused by in-plane seismic shear forces. The largest diagonal cracks were 3/4 inch wide (fig. 17) and the smallest were hairline. Distribution of forces and resulting cracking and displacement were unusual in that the smaller (more flexible) piers have the largest cracks. This might be explained by the differential ground movement and rupturing, which could induce local shear forces in piers that could not be redistributed by the continuity of the construction above.

The south wall piers would have had a relatively low (about 5 psi) shear stress under code force levels, based on uniform distribution and gross sections. This stress is based on horizontal acceleration of 13.3 percent of gravity of the mass of the building elements.

SEISMIC FORCE LEVELS

Seismic force levels are estimated to have been between 20 and 40 percent of gravity. Local ground rupturing and possible landslides contributed to the wide variance of response between individual buildings. Approximately 1 mile to the southwest true tectonic ruptures, identified as the Tujunga segment of the San Fernando fault zone, occurred at the surface. Approximately 20 inches of differential vertical movement was measured where the surface rupture crosses Little Tujunga Road.

GROUND RUPTURES

Numerous minor landslides occurred in the hills around the project site. The paved road at and near the building area had several surface ruptures, with up to a foot of vertical displacement; landslides caused fissures and embankment slumping of several feet. Ground cracks within the building area were widespread, resulting in severe sidewalk displacement, buckling, and other damage. (Note crack in asphalt paving, fig. 15.) Generally, the sloping site, with considerable cut and fill and coupled with strong basic ground motion, resulted in many ruptures and slides throughout the area.

COST ESTIMATE

The recreation building will require extensive repairs and/or replacement of the floor slab, walls, roof construction at the wall lines, sidewalk canopy, and numerous nonstructural items such as ceiling, finishes, and the like. The replacement cost of the building, prior to the earthquake, could be estimated at about $32 per square foot, or a total value of about $126,000. The cost of repairs would be about two-thirds of the building cost, or $84,000.

The dormitory building will require extensive re-

pair and/or replacement of the floor slab, portions of the south wall, and architectural items such as lights, finishes, and the like. The replacement cost of the building, prior to the earthquake, could be estimated at about $26 per square foot, or a total value of around $260,000. The cost of repairs would be about 40 percent of the total cost, or $100,000.

CONCLUSIONS

Damage to the recreation and dormitory buildings was fairly heavy, relative to what normally would be expected from an earthquake of this magnitude, the building code assumed level of safety, and the type of construction utilized. Structural damage to other buildings on the site was extremely light or nonexistent. The excessive damage to the two buildings can be explained, in part, by extensive ground faulting, unusually high ground accelerations, and possible general earthquake-effect amplifications associated with the cut-and-fill nature of the site.

Several observations were noted in the construction details that may be helpful in improving the performance level of this type of structure. Concrete block walls with cells filled only at reinforcing (24 in. apart) seemed to lack the normal strength and toughness of masonry construction. This was particularly true at the recreation building. Masonry piers with different types of units (block and brick) combined did not hold together well. Unusually large separations between outer and inner layers were noted.

Building with various combinations of seismic force-resisting systems that have different stiffnesses (walls, diaphragms, and steel frames) must be analyzed carefully to insure that stiffer elements are not required to rupture or fail before the more flexible elements can begin to resist their assumed loads. The failure of more rigid elements prior to the assumption of loads by other systems may not reduce necessarily overall building stability, but it can result in excessive damage to structural and nonstructural elements.

RECOMMENDATIONS

Engineering design and construction practices could be revised so that the resulting structures would be substantially more resistant to earthquake forces. The following suggested revisions will not make the resulting structure "earthquake proof," but they will increase the safety factor against earthquake damage.

1 Grout all cells solid in concrete block shear walls.

2 Provide more than the minimum amount of reinforcing steel, but in the form of bars, not wall mesh.

3 Prohibit the combination of significantly different types of individual masonry units in masonry shear walls.

4 Provide careful consideration of the relative stiffness of seismic force-resisting elements and systems, including consideration of their true rigidities and strengths, as opposed to arbitrary values assigned by the building code.

5 Provide additional strengthening where buildings are partially on fill material that may have a tendency to shift or slide during an earthquake.

Hospitals and Medical Facilities

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295 SUMMARY AND CONCLUSIONS FOR HOSPITALS AND MEDICAL FACILITIES

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The San Fernando earthquake was particularly destructive to hospitals and medical facilities. Most of the major structures in the heavily shaken area were medical facilities. Four major hospitals (Olive View, Veterans Administration, Holy Cross, and Pacoima Memorial Lutheran) were located within a radius of 9 miles of the epicenter. At the Veterans Administration Hospital (44), some of the buildings that were built prior to 1933 collapsed. The other three hospitals, which were built within the last 12 years with earthquake-resistant features, all suffered significant damage resulting in evacuation. There were, in addition, three medical office buildings (Foothill Medical Center, Pacoima Lutheran Medical Center, and Indian Hills Medical Center), two psychiatric units (Golden State and Olive View), and one mechanical equipment building (Olive View). All, except Golden State, were damaged significantly.

The structural damage descriptions and analyses in the following reports specifically point to many of the problems in conjunction with earthquake-resistant structures, including those of steel braced frames, concrete shear walls, inadequate separation between buildings, and irregular framing systems.

Medical Buildings Near Hansen Lake

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This group of four medical buildings located near Hansen Lake has been studied because of their contrasting performance in close proximity to the ground fault breaks and their relationship to the Olive View Hospital. The Foothill Medical Center is located at the corner of Van Nuys Boulevard and Dronfield Avenue. The other three buildings (fig. 1), located within a single block on Eldridge Avenue, are the Pacoima Lutheran Medical Center, the Golden State Community Mental Health Center, and the Pacoima Memorial Lutheran Hospital. Their locations also are shown on the building location map in the introduction to Building Reports.

Geologically, both sites are on alluvial deposits generally classified as unconsolidated gravel, sand, silt, and clay. The Foothill Medical Center is located approximately 8 miles from the February 9 epicenter and 1 mile south from the nearest fault break. The Pacoima complex is approximately 8½ miles from the epicenter and less than about ¼ mile south of the nearest fault break. Faulting generally has been referred to as thrust faulting, where one block of a formation moves up and over the other block. Both sites are located on the lower block.

It is interesting to note that these four structures, all designed by different architects and engineers at different times, performed quite differently. All designs were checked by the Los Angeles City Department of Building and Safety, and construction was inspected by the municipality as required by city ordinance. Three of the four projects included design and construction features that were not adequate under strict interpretations of the Los Angeles City

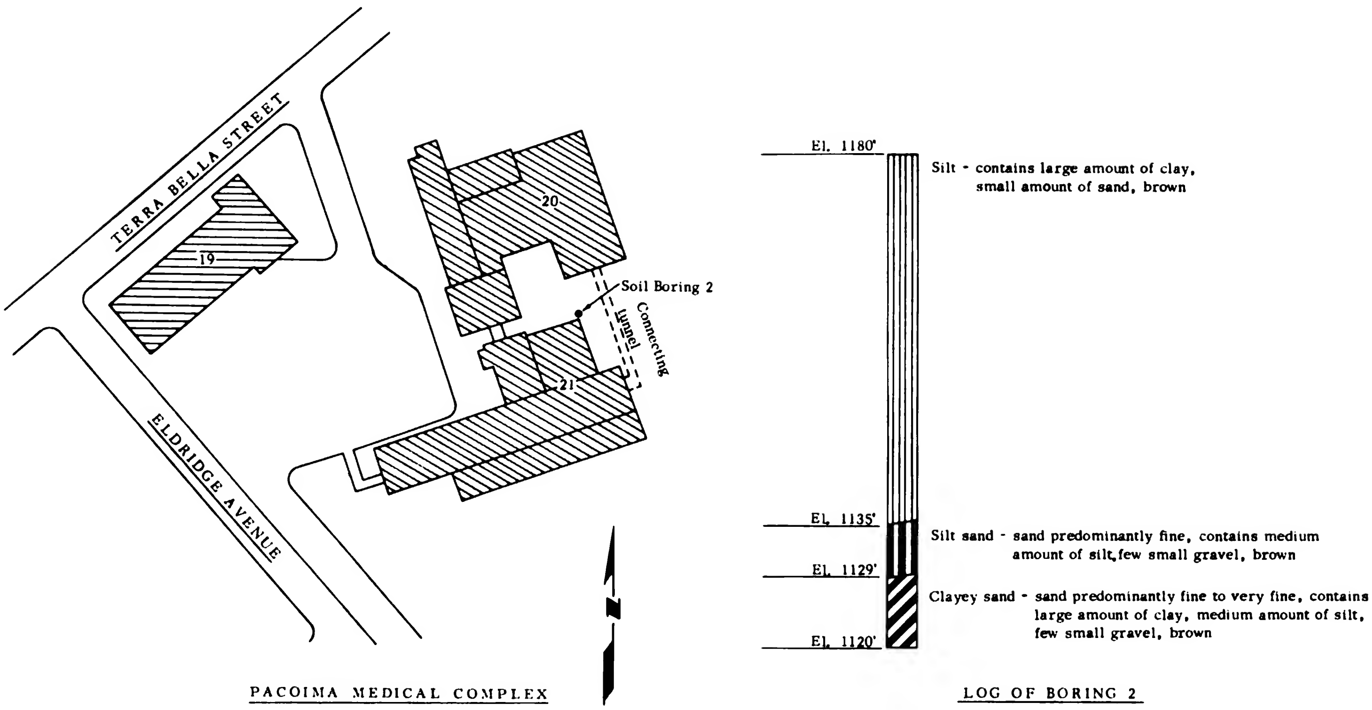

Figure 1.—Pacoima medical complex. Plot plan and boring log 2.

Building Code. These questionable features were directly responsible for the type of damage observed in the structures.

It is reassuring to note that on the fourth project, in which the code-required forces were carried from point of origin to point of resistance by a complete, simple, continuous stress path, no structural damage and little or no finish damage occurred.

Foothill Medical Center (18)

12502 Van Nuys Boulevard, San Fernando

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GENERAL DESCRIPTION

The Foothill Medical Center is a two-story rectangular structure with structural steel framing (fig. 1). There is a small penthouse but no basement. The drawings state that the building was designed to conform with the 1962 edition of the Los Angeles City Building Code. Construction took place during 1963.

The Foothill Medical Center is located on the southern corner of Van Nuys Boulevard and Dronfield Avenue. The soil at the site is described as silty sand, with some gravel, of low to medium density. A typical boring log is shown in figure 3c. Free ground water was not recorded in any of the borings reported in the original foundation investigation. The foundations are continuous strip footings on compacted fill and designed for an allowable bearing pressure of 2,000 psf. The first floor is a concrete slab on ground.

The second-floor framing plan is shown in figure 2. The roof framing is similar, except that the members are slightly lighter due to the lighter loads. The second-floor slab is a 3½-inch-thick lightweight concrete slab on corrugated metal forms and reinforced with 6- by 6-inch/No. 4 by No. 4 welded wire fabric. The slab is supported at 5-foot centers with open web joists spanning to the four longitudinal steel bents. The four longitudinal steel bents consist of a series of rigid frames, each consisting of two columns and connecting beam (fig. 3a). The beams connecting the rigid frames within a bent are simple beams spanning between hinges that are typically located 18 inches from the column centerline. The longitudinal beams are continuous through the joint with the columns welded and bolted to the beams for rigidity (fig. 3b). The structural steel was specified as ASTM A–7 on the drawings. The penthouse is 54 by 37 feet in plan and is framed with 2- by 10-inch roof

Figure 1.—Foothill Medical Center. View looking at southwest corner. Los Angeles City Department of Building and Safety, John Shadle, photographer.

joists supported on exterior 2- by 4-inch stud walls and steel beams on pipe columns in the center.

The exterior walls contain continuous horizontal windows at each floor around the entire building except at the bracing on line 11 (fig. 2). The spandrels above and below windows are stucco on wood studs, supported and braced by the steel framing. A pressed aluminum screen that acts as a sun screen is located around the perimeter about 4 feet from the exterior wall. This aluminum screen is supported by structural steel tubes and horizontal channels and is braced to the floor framing at the second floor and roof levels. This screen can be seen in figure 1.

Interior partitions consist of plaster on wood studs to the ceiling line. The ceilings are generally T-bar type suspended ceilings. The partitions along the first-story longitudinal corridor were braced with 2 by 4s from the ends of the lower chord of the open web joists. This bracing did not exist for the second-floor partitions.

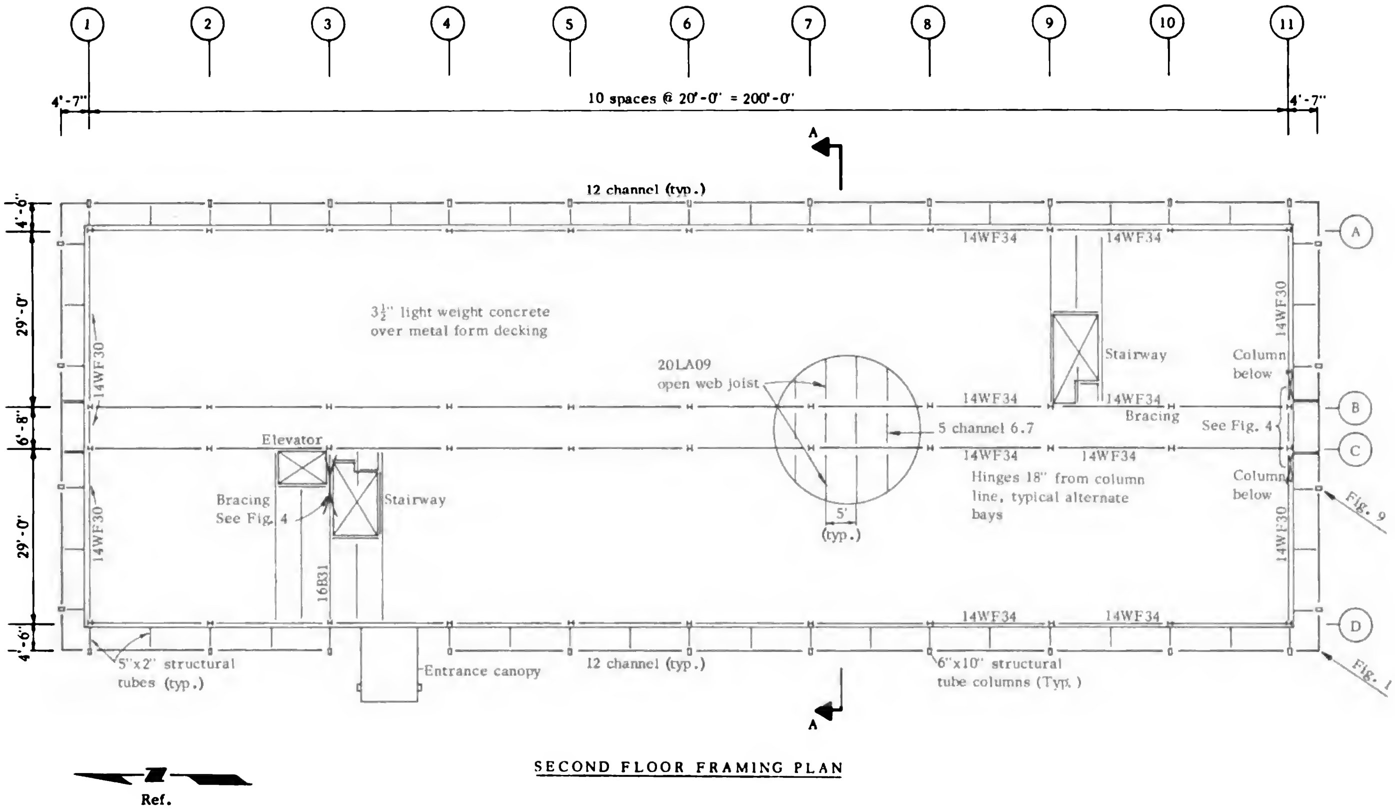
Design lateral forces were in accordance with the 1962 Los Angeles City Building Code. In the longitudinal direction the rigid frames resist the lateral forces, and the design is based on a seismic K factor of 0.67. In the transverse direction the lateral forces are resisted entirely by the diagonal bracing on lines 3 and 11 (fig. 2), as shown in figure 4. The transverse seismic forces were designed using a K factor of 1.33. The seismic coefficient C was 0.10 in both directions. Seismic forces govern over wind forces for design.

EARTHQUAKE DAMAGE

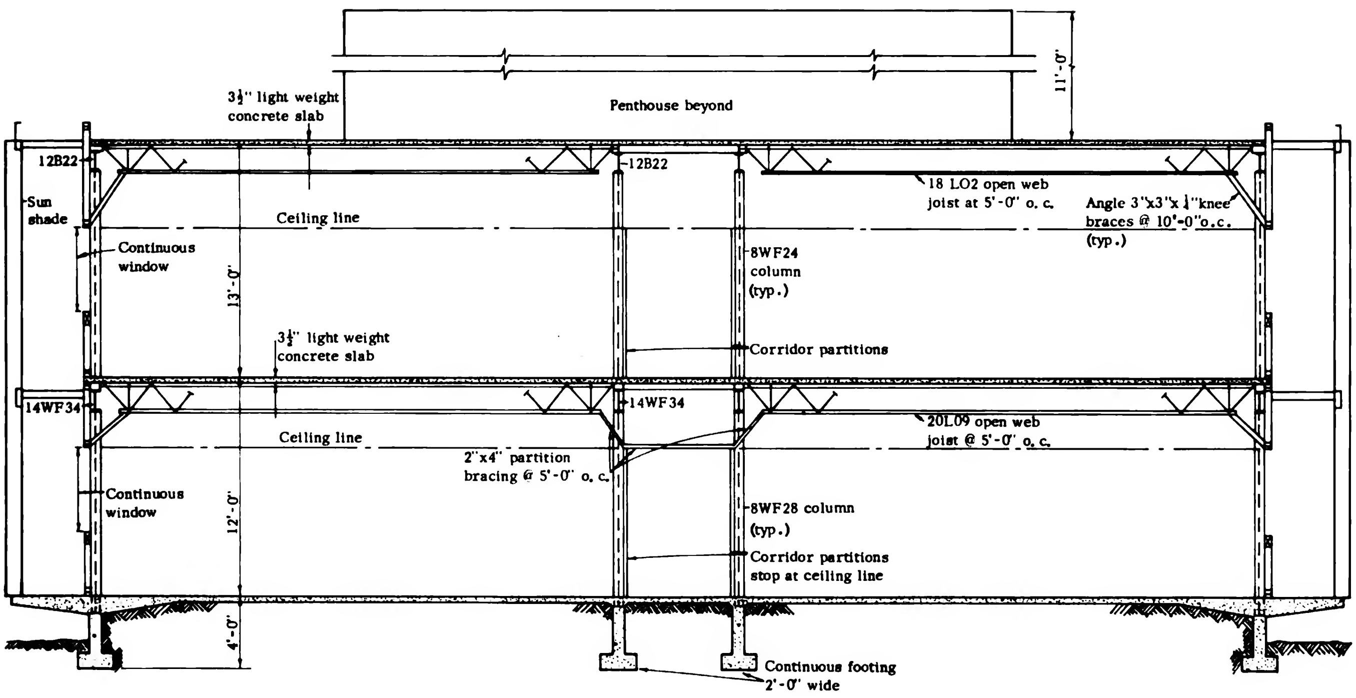
The principal structural damage was found in the diagonal bracing on line 3. The base of the 5WF16 column was bent severely (fig. 5). It should be noted that the diagonal brace at this one column was fabricated and installed so that the axis of the brace intersected the axis of the column approximately 11½ inches above the base plate instead of at the top of the base plate, as shown in detail 3 of figure 4. This column is further illustrated in figure 6 after it was removed from the structure. Note also in figure 5 the considerable spalling of concrete at the base of this column. The anchor bolts were still in place, although they may have been pulled out partially because of the bending of the column caused by the eccentric connection. The compression portion of this column bending caused the ½-inch-thick base plate to bend (fig. 6). Figure 7 shows the condition above at the second-floor framing where all bolts between the second-floor beam and the first-floor bracing were sheared. Note also the loose bolt at the bracing connection at the top of the beam and the yielding of the bottom of the web of the 16B31 near the sheared bolts. The stiffener plate shown on the drawings (fig. 4, detail 1) was omitted, perhaps owing to interference with the web connection of the beam framing the stairway opening. The connection at the second floor at line C (fig. 4, detail 2) also contained sheared bolts, as shown in figure 8. The drawings called for 6- by 6- by ¼-inch web clip angles with four ¾-inch bolts on each leg instead of the top and bottom flange connections provided. The clip angles were deleted and the beam flange connection, which was typical elsewhere on the job, was substituted. The second-floor bracing remained intact with only slight bending of the gusset plates at the connections.

The braces on line 11 also showed evidence of distress as a result of the earthquake, although no failures were observed. Figure 9, the exterior wall at the braces after the earthquake, shows some movements in all four columns that caused plaster damage. A few of the gusset plates showed slight signs of bending, but no other structural distress was observed on this line.

Other structural damage was confined primarily to the bases of the 6- by 10-inch structural tube col-



SECOND FLOOR FRAMING PLAN



SECTION A-A

Typical Building Section

Figure 2.—Foothill Medical Center. Second-floor framing plan and section A-A.

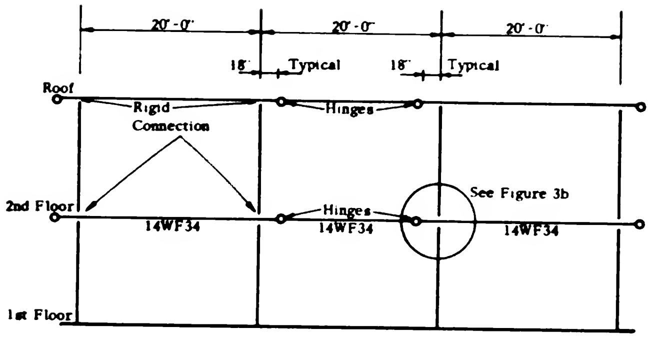


Figure 3a ELEVATION OF TYPICAL LONGITUDINAL RIGID FRAMES ON LINES A, B, C, & D

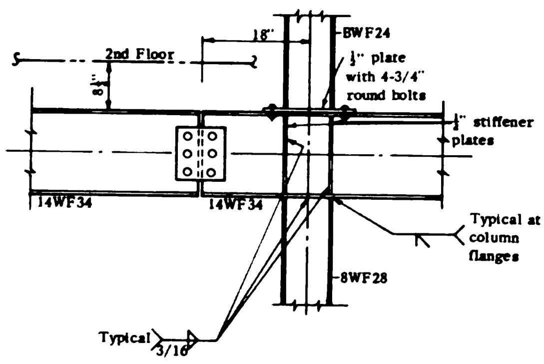


Figure 3b

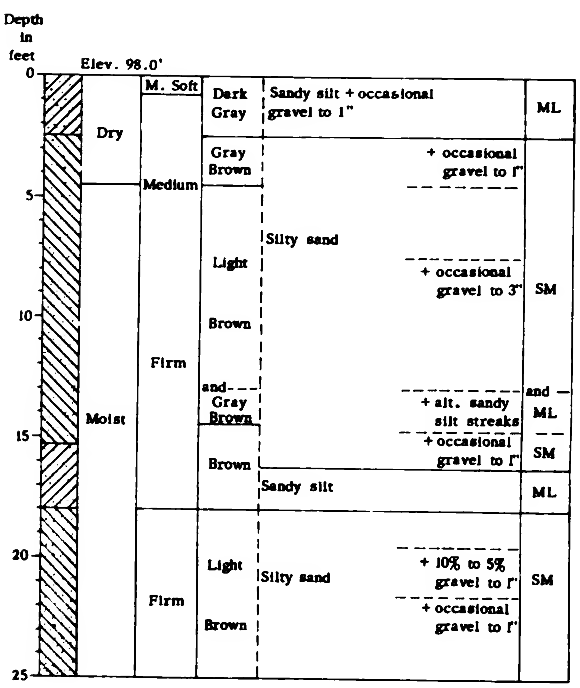


Figure 3c SUMMARY OF TYPICAL SOIL BORINGS

Figure 3.—Foothill Medical Center. Longitudinal frames and borings.

umns that support the aluminum sun screen. Concrete was spalled under the base plate of many of these columns (fig. 10). The anchor bolts were bent horizontally with very little cover, and the movement of the structure induced rotation at the column base, causing the concrete to spall. An angle was welded to the column and expansion bolts, shown in figure 10, to repair this damage.

There were significant cracking and damage to the interior partitions and the suspended ceiling system. The 2- by 4-inch braces to the first-story corridor partitions generally were split at the single toenail connecting them to the top plate of the partition (as shown on the building section, fig. 2). At several locations along the corridor the partition had separated from the column fireproofing, indicating considerable transverse movement of the frame. The plaster walls of the stairway near line 9 had considerable cracks in the first story, a further indication of the considerable transverse movements that took place. The metal runners of the T-bar ceiling system were buckled in numerous locations, generally at the cutouts where the hanging wires can be connected, because that location was the weakest spot. Extensive spalling and cracking of the exterior stucco occurred, especially at the corners, and considerable glass was broken. Figure 11 shows plaster and glass damage at the north corner.

The penthouse and mechanical equipment on the roof generally performed well, although the elevator was out of operation. Books stored in the penthouse were thrown all over the floor.

DISCUSSION OF DAMAGE

It is of interest to examine the design of the bracing system for the transverse lateral forces since its failure, which represents the major structural damage, permitted transverse movements resulting in glass and partition damage. Calculations based on the drawings and the 1962 Los Angeles City Building Code reveal the following results: The design forces to be taken by the bracing on line 3 would be approximately 63 kips at the roof and 80 kips at the second floor, resulting in a first-story shear of 143 kips on line 3. The diagonal braces have an l/r of about 190 to 200, based on overall length, or one-half that value if the diagonal in tension is assumed to brace the diagonal in compression. Building codes, design specifications, and authoritative text-

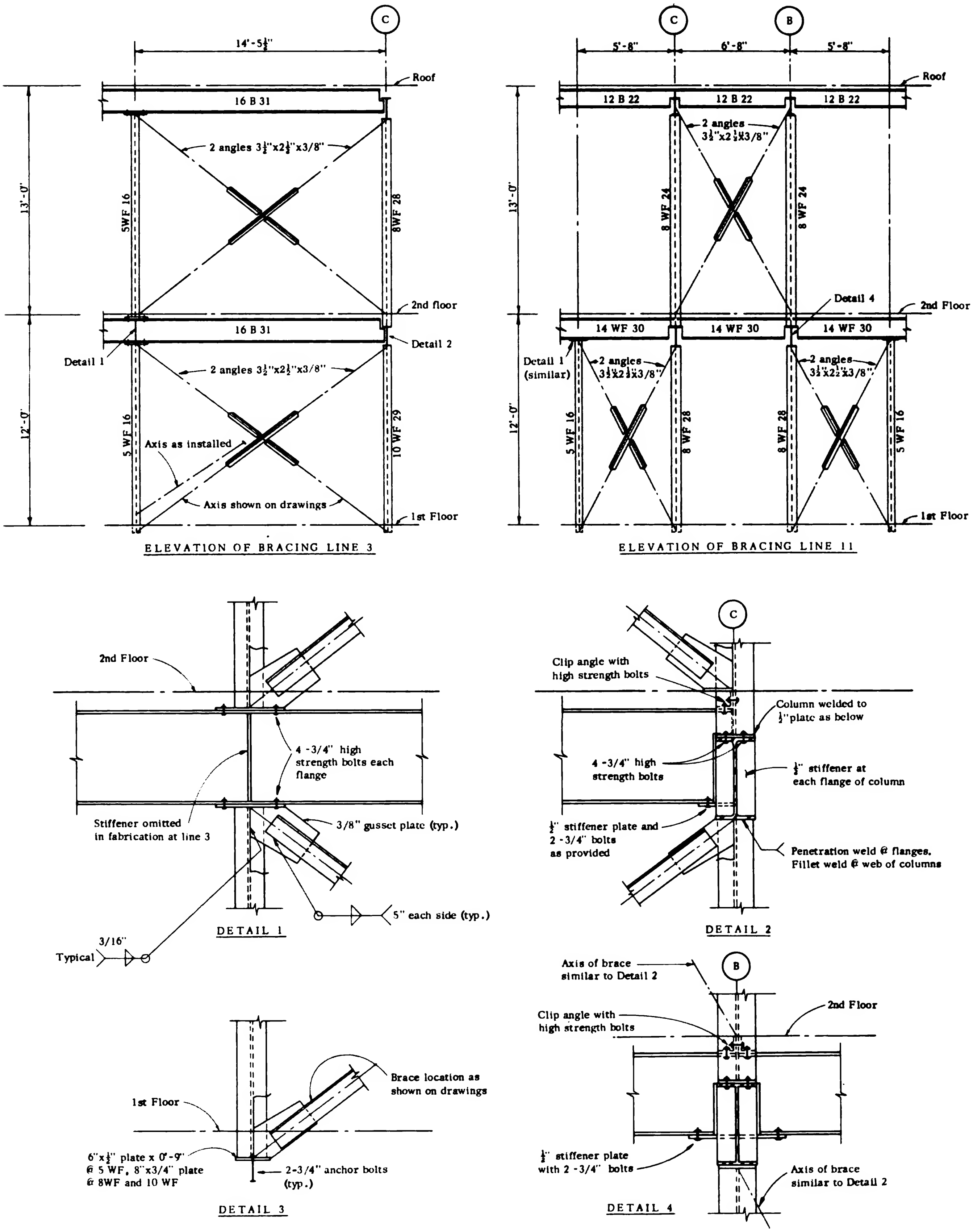


Figure 4.—Foothill Medical Center. Bracing elevations and details.

Figure 5.—Foothill Medical Center. Base of 5WF16 column on line 3 with diagonal brace connection. Note eccentricity of brace of 11½ inches. E. E. Schader photograph.

books are not clear as to the value that may be assigned to diagonals in compression in configurations and with $^{l}/r$, as used in these bracing elements. Many engineers neglect the compression strengths and consider the angle diagonals to work in tension only. Because the angles do have a substantial compressive resistance, the following analysis considers the stresses acting in both diagonals, equally in both tension and compression.

Under this assumption, the axial force in the first-story diagonals would be 88 kips when both diagonals are assumed to be resisting the shear. The capacity of two angles, 3½ by 2½ by ⅜ inch, is 113 kips in tension and 70 kips in compression ($^{l}/r$ of 96), with both values including the one-third allowable stress increase. The specified fillet welds at the ends of the braces, which showed no distress, have an allowable capacity of about 86 kips under the 1962

Figure 6.—Foothill Medical Center. Base of 5WF16 column on line 3 after being removed from structure. Gusset plate is up for reference. Los Angeles City Department of Building and Safety, John Shadle, photographer.

Figure 7.—Foothill Medical Center. Second-floor beam on line 3 with 5WF16 column and brace connections. Note missing bolt at lower flange and loose bolt at top flange. Loring A. Wyllie, Jr. photograph.

Uniform Building Code and about 114 kips under the 1970 Code. The shear capacity of the four ¾-inch round high-strength bolts in detail 1 of figure 4 is 52 kips with the one-third stress increase. A design force of 88 kips in the diagonal applied 11½ inches high on the 5WF16 column (fig. 5) would result in elastic stresses in the column over 200 kips per square inch, which is six times the yield stress. Overturning forces on line 3 resulting from the code level design forces would be 175 kips at the base neglecting frame interaction. The vertical dead load in the columns would be 50 kips or less. Two ¾-inch round anchor bolts were provided at each column base plate.

It would appear that the failure in the bracing on line 3 was initiated by yielding and slight initial deflection of the 5WF16 column near its base where the brace connection was eccentric. Once a small deflection of the columns took place, it caused virtually all the load to be transferred to the other diagonal, which resulted in the shearing of the bolts to the

Figure 8.—Foothill Medical Center. Connection underneath second floor on line 3 at line C. Note empty bolt holes to bottom flange of beam. Loring A. Wyllie, Jr. photograph.

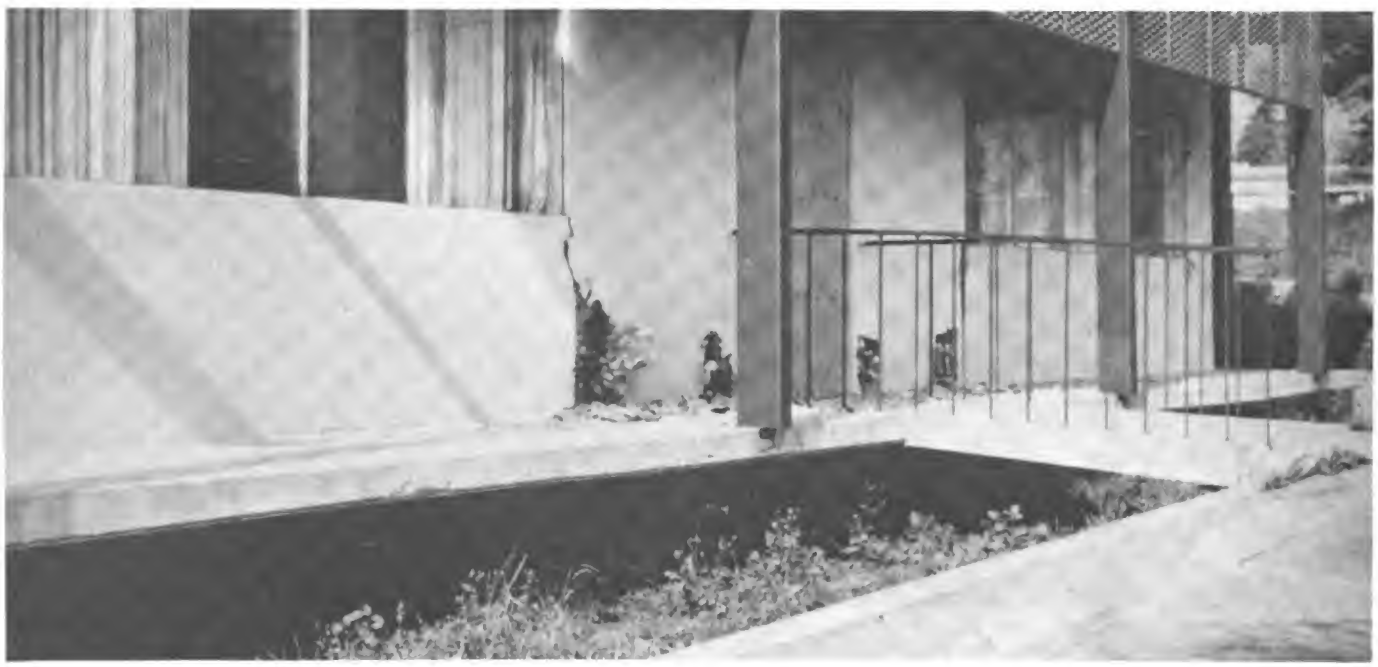


Figure 9.—Foothill Medical Center. Rear or south end of building, showing plaster damage at base of columns on line 11 at bracing. Loring A. Wyllie, Jr. photograph.

Figure 10.—Foothill Medical Center. Base of typical 6- by 10-inch structural tube column for exterior sun shade. Note spalled concrete and horizontal anchor bolts. Angle added after earthquake as a repair. Paul F. Fratessa photograph.

Figure 11.—Foothill Medical Center. Exterior of building at north corner showing plaster and glass damage. Loring A. Wyllie, Jr. photograph.

bottom flange of the second-floor beam. After the failure of these bolts, only one diagonal was available to work in both tension and compression and to resist the entire lateral load. This diagonal was connected to the 10WF29 column at the second floor, which in turn was welded to the longitudinal 14WF34 beam, thereby providing a connection even after the two bolts to the bottom flange of the 16B31 sheared (fig. 8). The sizable axial forces in the diagonal, coupled with the fabricated eccentricity at the 5WF16 column, caused the severe distortion of that column. It is estimated that a ground acceleration of approximately 7 percent of gravity would be sufficient to cause the first failure of the bracing system as fabricated and built, as compared to a code-required base shear of 13.3 percent of gravity. It appears that the first-story partitions tried to assist the failed bracing system. Forces apparently were transferred through the 2- by 4-inch struts to the corridor partitions to the capacity of the toenailing. Large transverse movements of the building could be verified by several interior columns that punched through the corridor partition, the severe cracking of the southern stairway and the extensive glass breakage at the north end, and the rotation of the exterior tube columns supporting the sun shade. This damage was most evident at the north end of the building where the bracing failed and permitted larger movements.

The diaphragm appears to have adequately performed its function in distributing forces to the bracing. There was no evidence of diaphragm distress. Continuous chords were provided and studs were located on top of the beams on lines 3 and 11 to transfer shears out of the concrete diaphragm and into the beam of the bracing system.

REPAIRS

Repairs to the structure consisted of replacing the sheared bolts and installing an oversized gusset plate at the first floor to partially compensate for the eccentricity of the one diagonal on line 3. Figure 12 shows this connection as repaired and can be compared with figure 5. A new 5WF column was installed on line 3 to replace the badly distorted columns. Angles were installed to reanchor the exterior screen support columns as shown in figure 10. Plaster, ceiling, and stairway repairs also were required.

Figure 12.—Foothill Medical Center. Base of 5WF16 column on line 3 as repaired with oversized gusset plate. Los Angeles City Department of Building and Safety, John Shadle, photographer.

Actual repair costs were not made available for this report; however, it appeared that damage amounted to approximately 10 percent of the value of the building prior to damage.

SUMMARY AND CONCLUSIONS

The Foothill Medical Center is a two-story light steel frame structure, 200 by 65 feet in plan. The longitudinal direction was braced with rigid frames which performed satisfactorily. The transverse direction was braced with two sets of X-bracing, of which one failed and caused considerable movement of the structure and nonstructural damage. The failed X-bracing had an eccentricity at one column base, which resulted in considerable bending of the steel column. The bolted second-floor connections of the X-bracing failed in shear due to the earthquake activity.

RECOMMENDATIONS

As a result of the observations of the earthquake performance of this building, certain general recommendations can be made.

1 The design engineer who knows the design concept of the building should be required to furnish construction support services to insure that the intent of the design is carried out in the field.

2 In order to insure adequate ductility, member connections in trusslike members such as diagonals and columns that form a part of the bracing system should develop the member. Building codes should reflect this requirement.

3 Codes and specifications must be more specific as to the amount of compression that can be permitted in members of the proportions used herein. It is recommended that the tension diagonal be considered as insufficient bracing to support the compression diagonal.

4 More care is required in major stress transfers to see that all stresses are accounted for. Shear transfers and uplift base plate connections must be adequate to transfer design stresses to the foundations.

Pacoima Lutheran Medical Center (19)

11600 Eldridge Avenue, Pacoima

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GENERAL DESCRIPTION

The Pacoima Lutheran Medical Center (fig. 1) is a two-story structure with penthouse. It is located on Eldridge Avenue at Terra Bella Street in the same block as the Pacoima Memorial Lutheran Hospital. This structure, built in 1964, has grouted brick shear walls, steel framing and bracing, and wood joists and floors. It was designed under the provisions of the 1963 Los Angeles City Building Code.

Although other buildings on this site used drilled-in-pile foundations, the light loads of this structure permitted the use of spread footings on a minimum thickness of 3 feet of compacted fill using a design soil pressure of 1,500 psf. The first-floor slab is 4-inch-thick slab on grade. Walls are supported by continuous footings.

The general construction materials for the floor and roof consist of plywood sheathing and wood joists supported on steel beams, which in turn are supported on steel tube columns. Intermittent reinforced grouted brick wall panels line the first-floor corridor to act as shear walls for longitudinal (east-west) loads, and vertical supports are used for some of the interior columns and beams. First-floor exterior walls generally are of glass with reinforced grouted brick sill walls 5½ feet high. Second-floor walls are of wood framing.

The foundation and first-floor plan and second-floor framing plan are shown in figure 2. The roof and penthouse floor plan in figure 3 and the typical cross section of the building at the interior brick walls are shown in figure 4, section A–A.

As shown in the figures, the second floor consists of 1½ inches of lightweight concrete fill on ⅝-inch plywood, supported on 2- by 4-inch Douglas Fir wood joists at 16 inches on center spanning 20 feet. These are supported typically by 18WF45 steel beams supported by 3- by 5- by $\frac{5}{16}$-inch steel tube col-

Figure 1.—Pacoima Lutheran Medical Center, looking west after repairs. Loring A. Wyllie, Jr. photograph.

umns. At the corridor, joists change direction with 2- by 6-inch joists at 16 inches on center spanning 6 feet.

Roof framing consists of 5/8-inch plywood sheathing supported by 2- by 10-inch joists at 16 inches on center spanning 20 feet, and supported by 12WF58 steel beams. Steel beams span 27 feet from 3- by 5- by 3/16-inch steel tube columns at the exterior walls (fig. 3, lines A and F) to corridor steel tube columns 3- by 5- by 3/16-inch (fig. 3, lines C and D). The mechanical penthouse is sunk below the gabled roofline (fig. 4) and consists of a 4-inch concrete slab on 2- by 10-inch wood joists that span across the width of the penthouse to 12WF27 steel beams. These beams are hung from the typical 12WF58 roof beams.

Reinforcing steel is indicated on the drawings as ASTM A–15; the specified ultimate concrete compressive strength is 2,000 psi; structural steel is specified as ASTM A–36; pipe columns are specified as ASTM A–53 grade B; timber is specified as construction grade; and brick is specified as 2,500 psi units without continuous inspection.

Figures 1, 4, and 5 show the continuous band of high windows to be typical around the perimeter of the building at both the first and second stories. The "sill" in the first story is of grouted brick, and the framing at the second story is of wood. Both of these sill elements provide lateral support to the steel tube columns.

A study of the plans (figs. 2 and 3) shows the building to be a rectangle at the roof level, but with a "hammerhead" at the east end below the second floor. Grouted brick shear walls at the outer faces of the hammerhead are the stiffest elements in the east-west direction below the second floor. This configuration requires a major transfer of lateral loads by the second-floor diaphragm.

The plywood diaphragms on this building are fully blocked, and the nailing is adequate for the code-required forces. Chord ties are provided at the second floor by double joists that are spliced at each column by bolting to steel plate ties. These, too, are adequate for code-required forces. The roof diaphragm is pierced by the mechanical room. At the east end of the mechanical room struts are provided to reinforce this "notch," but struts are not evident at the west end.

The seismic resistance of this building is provided by plywood diaphragms and a combination of brick and plywood shear walls and steel X-bracing.

East-west (longitudinal) lateral forces are resisted by the following system. Between the second floor and the roof, the total lateral force system is composed of plywood panels on each side of the central corridor. Shear collectors for this system depend on the double wood plates at the top of the wall. Details for transfer of overturning forces to the walls below were not determined. There were no exterior shear elements above the second floor in this direction. However, the exterior tube columns and the exterior sill walls were reported to have been designed for 25 percent of their tributary lateral load.

Below the second floor, there are four 5-foot-long 10-inch-thick reinforced grouted brick shear walls on each side of the central corridor, in the same plane as the plywood walls above.

The first story contains 22-foot-long 10-inch-thick reinforced grouted brick walls at the ends of the hammerhead, lines A′ and G. The longitudinal lateral forces of the main portion of the second floor are transferred to these walls by horizontal rod bracing, anchored to double 2- by 14-inch ties on lines A and F. Additional resistance is provided by the steel tube columns that cantilever a short distance out of the continuous 5½-foot-high reinforced grouted brick "sill" wall, as shown in figure 4, section A–A.

North-south (transverse) lateral forces are resisted by the following system. There are two shear elements in the north-south direction—on the exterior west wall at line 1 and adjacent to the elevator at line 8.4 (fig. 6). In both cases, shear walls of reinforced grouted brick are provided in the first story, and 7-foot-wide steel X-braced panels are placed between the second floor and the roof. Elevations of

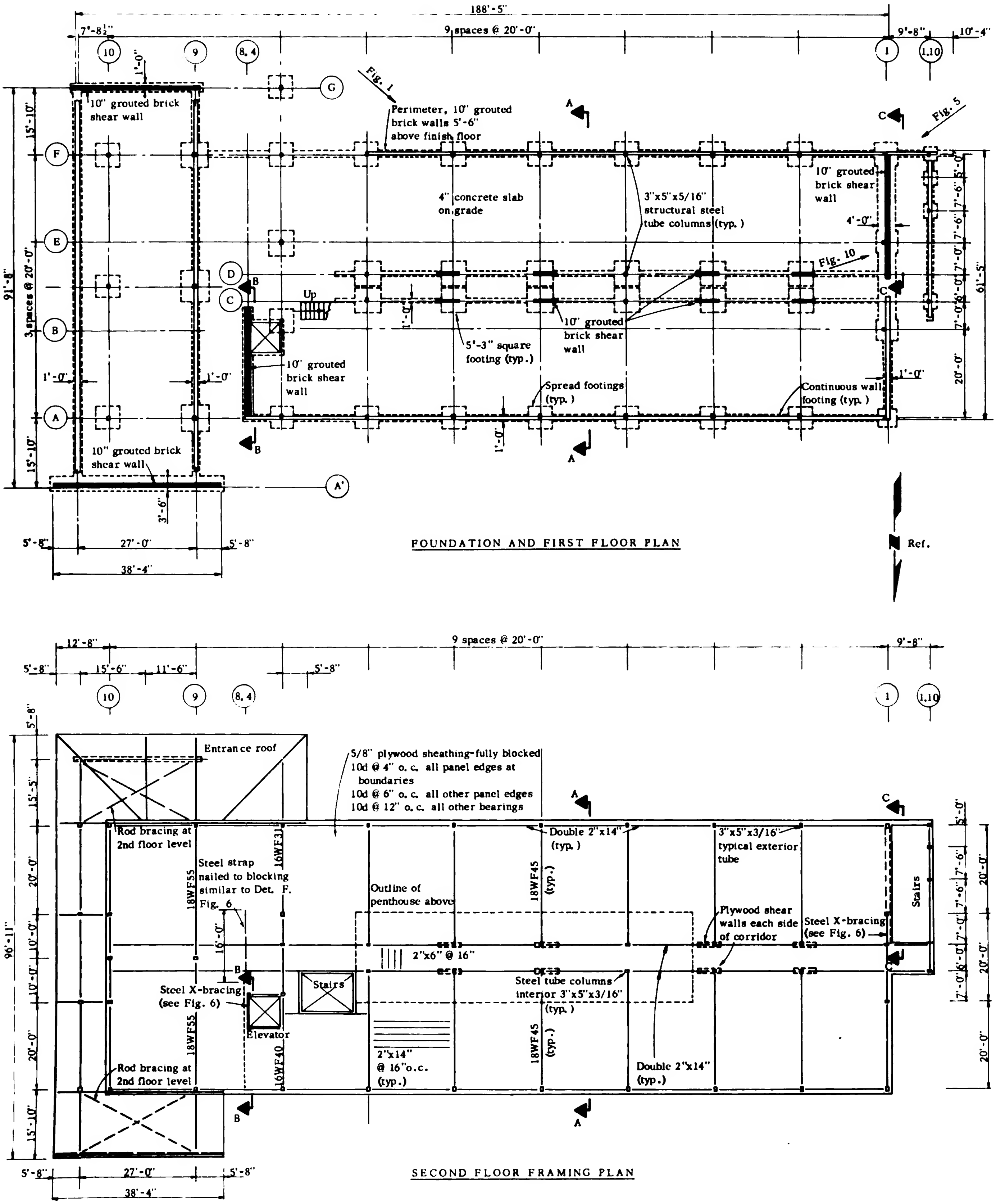

Figure 2.—Pacoima Lutheran Medical Center. Foundation and first-floor plan and second-floor framing plan.

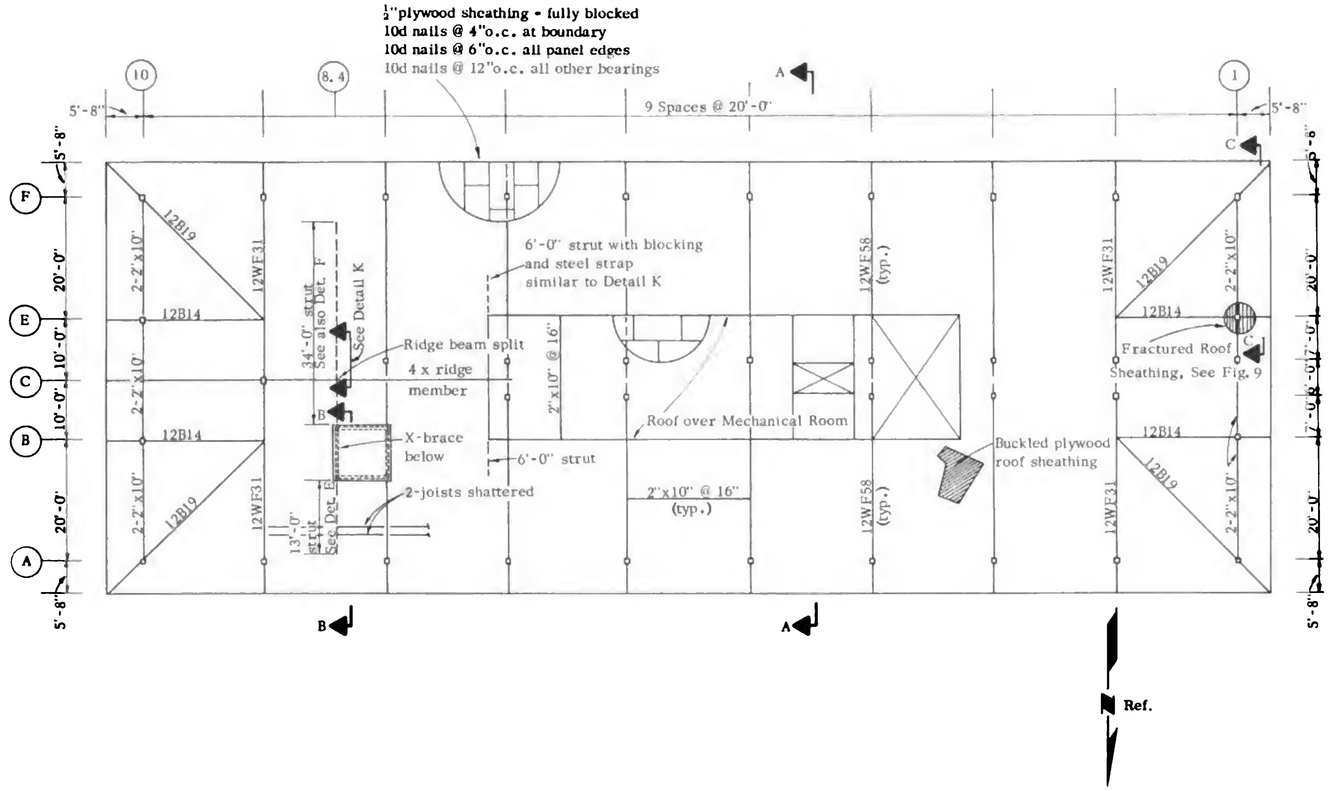

Figure 3.—Pacoima Lutheran Medical Center. Roof framing plan.

these walls and the pertinent details are shown in figure 6.

EARTHQUAKE DAMAGE

An initial examination of the exterior of the building led to the misleading conclusion that there was little or no structural damage. Figure 5, a photograph taken immediately after the earthquake, shows a few broken windows, but little else in the way of damage. It was reported that office equipment, desks, and filing cabinets moved across the second floor and, in some cases, tipped over. Some nonstructural damage was reported, such as buckling of the second floor **T**-bar ceiling, partition damage at the second floor, and spalling of the first-floor brick sills.

Although the building survived the earthquake with a relatively moderate amount of total damage, there was significant structural damage in several key areas. Probably the most significant occurred at the north-south shear walls on lines 1 and 8.4 (fig. 6).

At line 1, the double 2- by 10-inch roof collectors (fig. 6, details A and B) failed at the connections to the steel **X**-bracing system at lines E and D (fig. 7). The tube column at line E was bowed in the direction parallel to the wall. The corner column at line F also yielded in bending, as shown in figure 8. It is evident that these columns acted as cantilevers above the window sill, probably after the buckling failure of the steel plate **X**-braces. In figure 6, detail B, there is considerable eccentricity involved in the transfer of shear from the roof diaphragm into the **X**-bracing system. The resulting moment must be transferred through the straps and bolts to the double 2- by 10-inch wood members. It is evident that this contributed to the shattering of the wood members, which was probably followed by the buckling on the **X**-braces and the bending yield of the columns. Plywood roof sheathing above this connection buckled, causing roof failure (fig. 9). Below the second floor, the grouted brick wall cracked (fig. 10) where the tube column at line E was embedded in the wall (fig. 4, detail J).

At line 8.4, several failures occurred in the collector system used to transfer the forces to the steel **X**-bracing. This system is as shown in figure 6, details E through H. The 1-inch by $\frac{3}{16}$-inch by 34-foot strap noted in detail F extends to the ridge and beyond, as shown on the roof plan and figure 6, detail K. The 4- by 10-inch wood ridge member was split

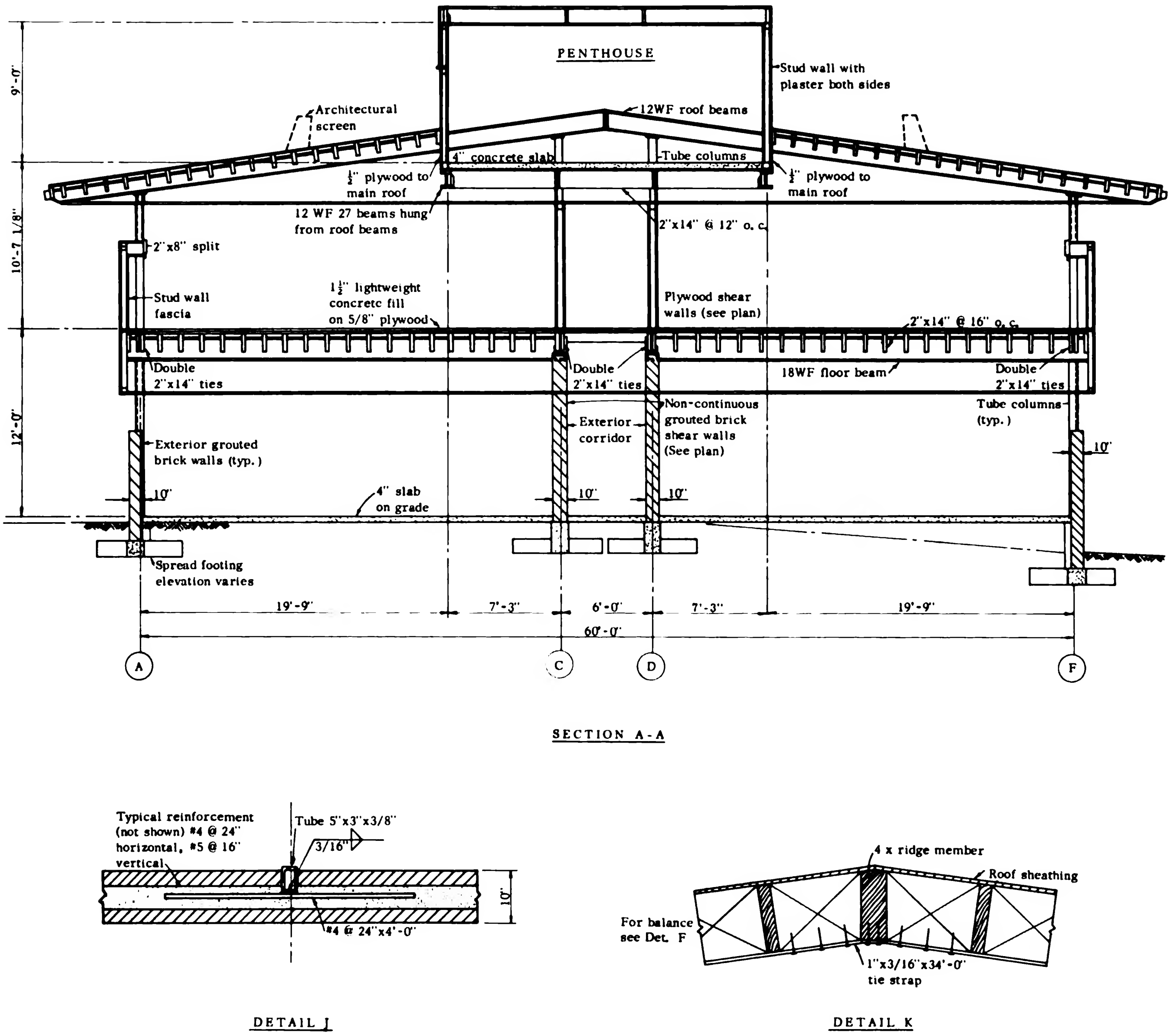

Figure 4.—Pacoima Lutheran Medical Center. Section and details.

badly at the strap connection. There was an area on the north side of the **X**-brace where blocking rotated in a vertical plane, adjacent joists shattered, and the strap buckled. The 1-inch by $\frac{3}{16}$-inch by 13-foot steel strap shown in detail E failed at the connection to the column, probably at the fillet weld. The **X**-bracing itself was not damaged. At the strap connection to the brick wall under the second floor (fig. 6, section B–B), nails from the strap to the blocking were popped out for a distance of several feet.

As shown on the roof framing plan, figure 3, the plywood roof sheathing buckled near the northwest corner of the mechanical penthouse. As noted pre-

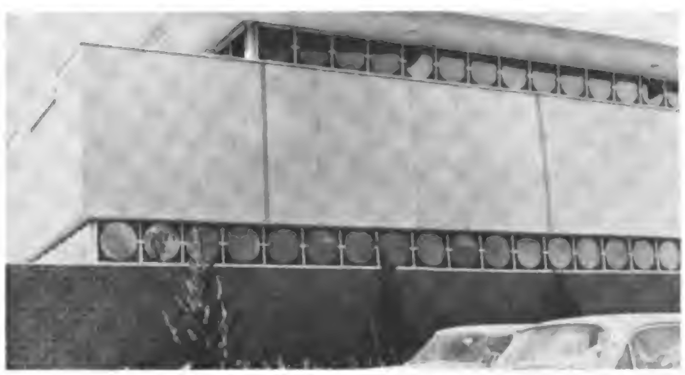

Figure 5.—Pacoima Lutheran Medical Center. Southwest corner, just after earthquake. Note two broken panes of glass at the second floor. Ralph Goers photograph.

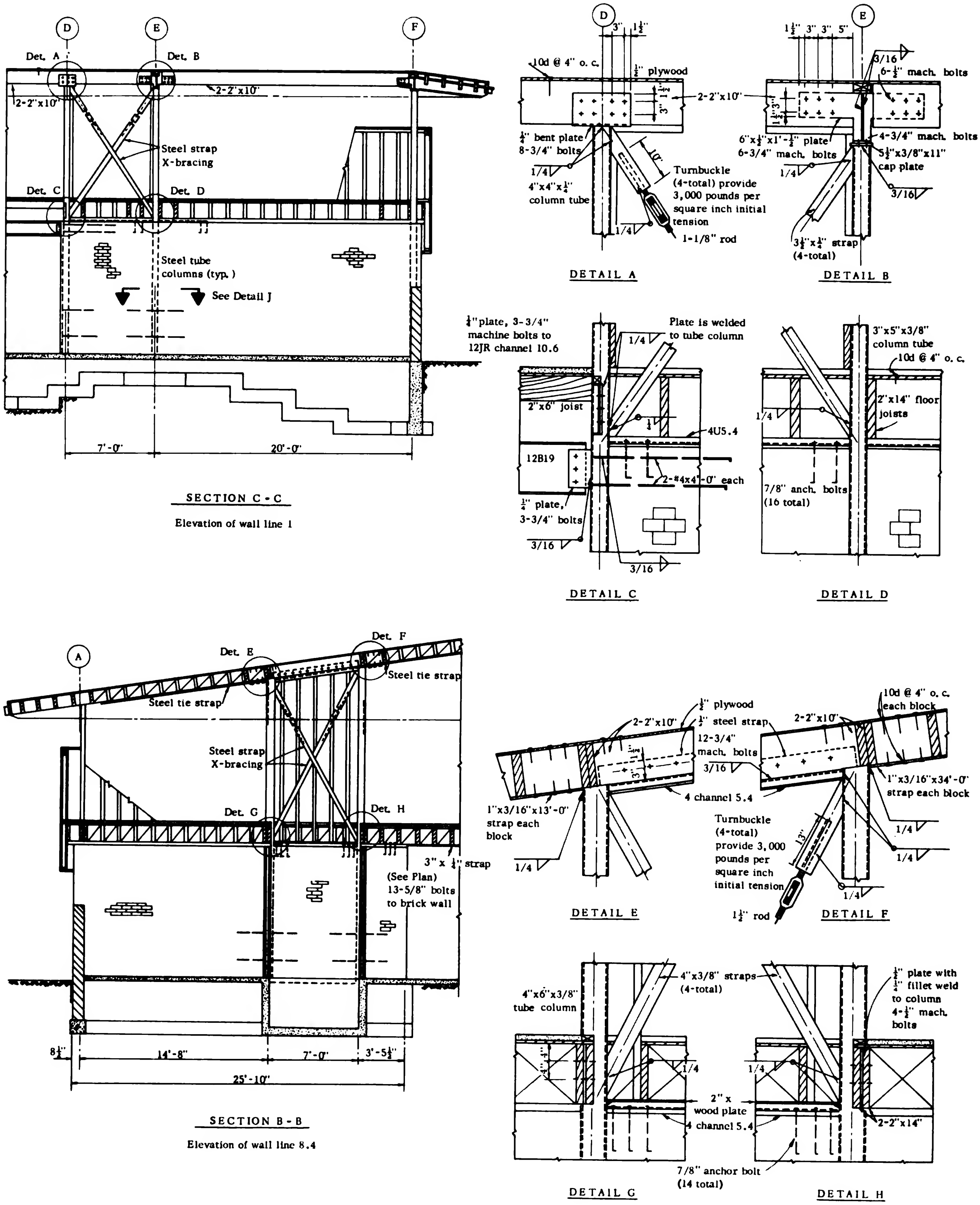

Figure 6.—Pacoima Lutheran Medical Center. Elevation of wall lines 1 and 8.4.

Figure 7.—Pacoima Lutheran Medical Center. Connection of steel bracing (line 1) to timber ties at roof on line E. Arrows point to splitting of 2- by 10-inch ties on each side of column. Ralph Goers photograph.

Figure 8.—Pacoima Lutheran Medical Center. Column F-1, second floor, looking south. Ralph Goers photograph.

viously, there was no north-south "chord" connection at this point. The horizontal **X**-bracing to the first-story walls at lines A′ and G performed adequately, although it was reported that there was some damage to the double 2- by 14-inch wood tie at column F–9, as well as splitting of the 2- by 14-inch ties at the bolted connections down half the length of the building.

There were several areas of damage in the walls themselves. In addition to the previously discussed cracking at column E–1, there was some spalling of the brick at the top of the first-story sill wall owing to the cantilever action of the embedded steel tube columns (fig. 11).

There also was damage at the second-story window sill in both the north and south walls where the 2- by 8-inch wood sill members split at the bolted connections to the columns. Again, this is indicative of the action of the columns in attempting to cantilever above this wall.

It was reported that the corridor plywood wall panels in the second story performed well, with only

Figure 9.—Pacoima Lutheran Medical Center. Break in roofing at plywood failure at southwest corner immediately above detail B (fig. 6). Ralph Goers photograph.

minimal damage, while the grouted brick walls below were sound.

REPAIRS

The building was under repair at the time of examination. Repairs consisted of strengthening the north-south shear walls and their collecting systems and adding new plywood panels in the north and south walls of the second story (lines A and F).

On line 1 (fig. 6), the **X**-bracing in the second story was replaced with a 15-foot-long grouted brick panel and a structural steel collector system at the roof. The cracked area in the first story was repaired by removing and replacing a 2-foot-wide band centered on the crack.

On line 8.4 (fig. 6), the collector system was strengthened by using a 3- by 3- by 1/4-inch steel angle strut collector connected to the diaphragm and the **X**-bracing system. A steel (8B10) ridge member was added perpendicular to the collector in this bay to resist the vertical component which will occur at this point. In the north and south exterior walls, double sheathed 1/2-inch plywood panels 5 feet long were added from the top of the first-story brick "sill" wall up to the roof framing. Connections were provided at both the roof and second floor to transfer the loads to these panels. Eight panels were added in each wall, centered on the column lines.

The total cost of repair work, including nonstructural repairs and some remodeling work, is estimated at $225,000. The original construction cost was $637,000.

SUMMARY AND RECOMMENDATIONS

The damage to this structure is indicative of the problems that can occur in a building constructed with several different materials, which requires great attention to details used to tie the various elements together. It also reflects the problems imposed by architectural restrictions, which may limit the size, number, and location of seismic-resisting elements. When these elements are small in number and size, the high concentration of forces can be extremely difficult to handle, and damage in one element or connection can lead to subsequent failures in other members.

Figure 10.—Pacoima Lutheran Medical Center. Grouted brick wall at first floor, line E, from inside. Note diagonal crack in brick. Ralph Goers photograph.

Figure 11.—Pacoima Lutheran Medical Center. Grouted brick wall at first floor structural tube column. Note spalled concrete at top of wall. Ralph Goers photograph.

As a result of the observations and analysis of building damage, several recommendations can be made.

1 Positive chord ties and collector members must be provided at all openings, discontinuities, and resisting elements. Plywood diaphragms without specific, substantial chords (as in the roof of this structure) are not adequate within themselves to resist the resultant concentrated stresses. Present building codes are not specific on this point.

2 Secondary stresses caused by eccentricities are of primary importance in considering the complete stress path between point of load and resistance.

3 Where members carrying tension or compression turn corners, as the strap of detail K, figure 4, then a transverse component of the force is introduced at the bend. This component must be provided for.

4 Where shear forces are transferred through wood blocking, each block acts as a diaphragm and must develop shears on all four boundaries. In wood construction, this is accomplished by end nailing the block through adjacent joists, sheathing, or ties, and where necessary, by toenailing the blocking to the joists. This nailing must be adequate to transfer the loads.

5 In order to provide adequate ductility to compensate for the disparity between code-specified and actual forces, connection details must be adequate to develop the main members, such as the steel **X**-bracing used in the second floor of this structure.

6 Architects must provide room and facilities for an adequate lateral force-resisting system. Structural engineers, at best, can alleviate only slightly an inappropriate basic layout. If adequate earthquake resistance is to be achieved, architects in California cannot afford the luxury of simulating the open and free designs so popular with many Eastern and Midwest architects.

7 Building officials and other review agencies must look beyond the letter of the building codes to enforce the intent of the technical provisions to achieve a reasonable, consistent, and continuous stress path from points of lateral loading to resisting element.

Golden State Community Mental Health Center (20)

11600 Eldridge Avenue, Pacoima

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GENERAL DESCRIPTION

The Golden State Community Mental Health Center is located immediately adjacent to the Pacoima Memorial Lutheran Hospital. Site conditions are described under the introduction to these facilities, and figure 1, section A–A, shows soil borings and pile depths for the Mental Health Center and the hospital.

The Mental Health Center, which was designed in 1968, is a two-story reinforced concrete and reinforced brick building with a partial basement in the east portion. The building is structurally separate from the hospital building, but, as shown in figure 2, access between the two buildings is provided through a connecting tunnel at the southeast corner and a passage at the southwest corner of the Mental Health Center. Figure 1, section B–B, shows a cross section through the building, indicating the relationship of the various levels.

Construction of the Mental Health Center had not been completed entirely at the time of the earthquake, and the building was not occupied. Earthquake damage was very minor, and construction was rushed to completion in order to fill the need caused by the evacuation of the hospital building.

The building is supported on a foundation system of 24-inch, round, drilled concrete piles, generally carried to a depth of 20 feet below the bottom of the pile caps. At this depth, the design pile capacity was 63.5 tons. Design capacity varied with length of pile to a maximum of 100 tons at 30-foot embedment. Because the piles were drilled to a predetermined length, the piles in the basement area extend to an elevation approximately 14 feet deeper than in the remainder of the building (fig. 1, section A–A). It is interesting that this pile capacity is approximately twice as great as the capacity for similar piles used on the main hospital building. All pile caps are connected with grade beams, and the ground floor is a

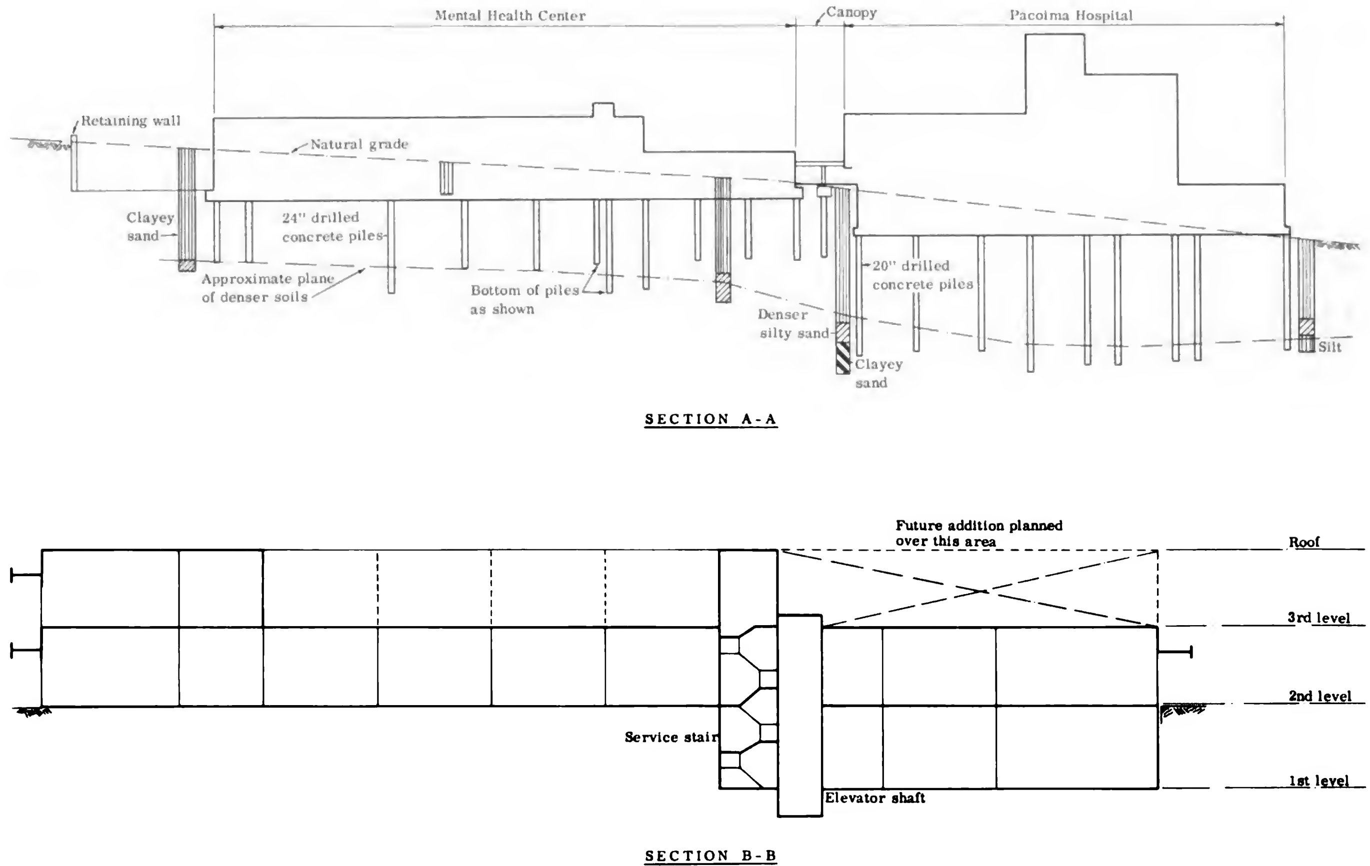

Figure 1.—Golden State Community Mental Health Center. Sections A–A and B–B.

typical concrete slab on ground. It is not designed to span between piles.

Figure 2 shows the typical floor framing to be reinforced concrete beam and slab, with a 7-inch-thick two-way slab covering most of the area. This system is supported by reinforced concrete columns, typically 14- by 14-inches square with tied reinforcement.

Exterior walls are generally of reinforced grouted brick. The west wall is shown in figure 3. The east and west walls are of brick piers, approximately 6 feet wide at 24 feet on center, with window wall between. The piers are connected with concrete spandrel beams above the window wall. In the north and south walls, the brick panels are generally longer in length. These walls are shown on the framing plan in figure 2. Typically, the grouted brick walls are 9½ inches thick and are reinforced with No. 4 bars at 18 inches on center each way in the center of the wall, with dowels to match to the foundations and slabs. Two No. 8 continuous vertical trim bars were provided at all wall edges. Continuous special inspection of the grouted brick work was required.

In addition to the grouted brick exterior walls, there are reinforced concrete walls around two elevator shafts (fig. 4). The concrete walls are 10 inches thick and are reinforced with No. 4 bars at 16 inches on center each way and each face. Basement walls and retaining walls are also of reinforced concrete.

The building was designed as a shear wall structure for resistance to earthquake forces, with a design base shear equal to 13.3 percent of gravity. Loads were distributed to the wall system, with resulting maximum design shears of 41 psi on the grouted brick and 83 psi in the concrete. Performance of the system was excellent.

The tunnel connecting this building with the main hospital was structurally separate from both buildings. The tunnel is a complete reinforced concrete box with the floor slab as the foundation. Piles were *not* used to support the tunnel.

EARTHQUAKE DAMAGE

There was very little damage to this building, and there is evidence that the damage that did occur may

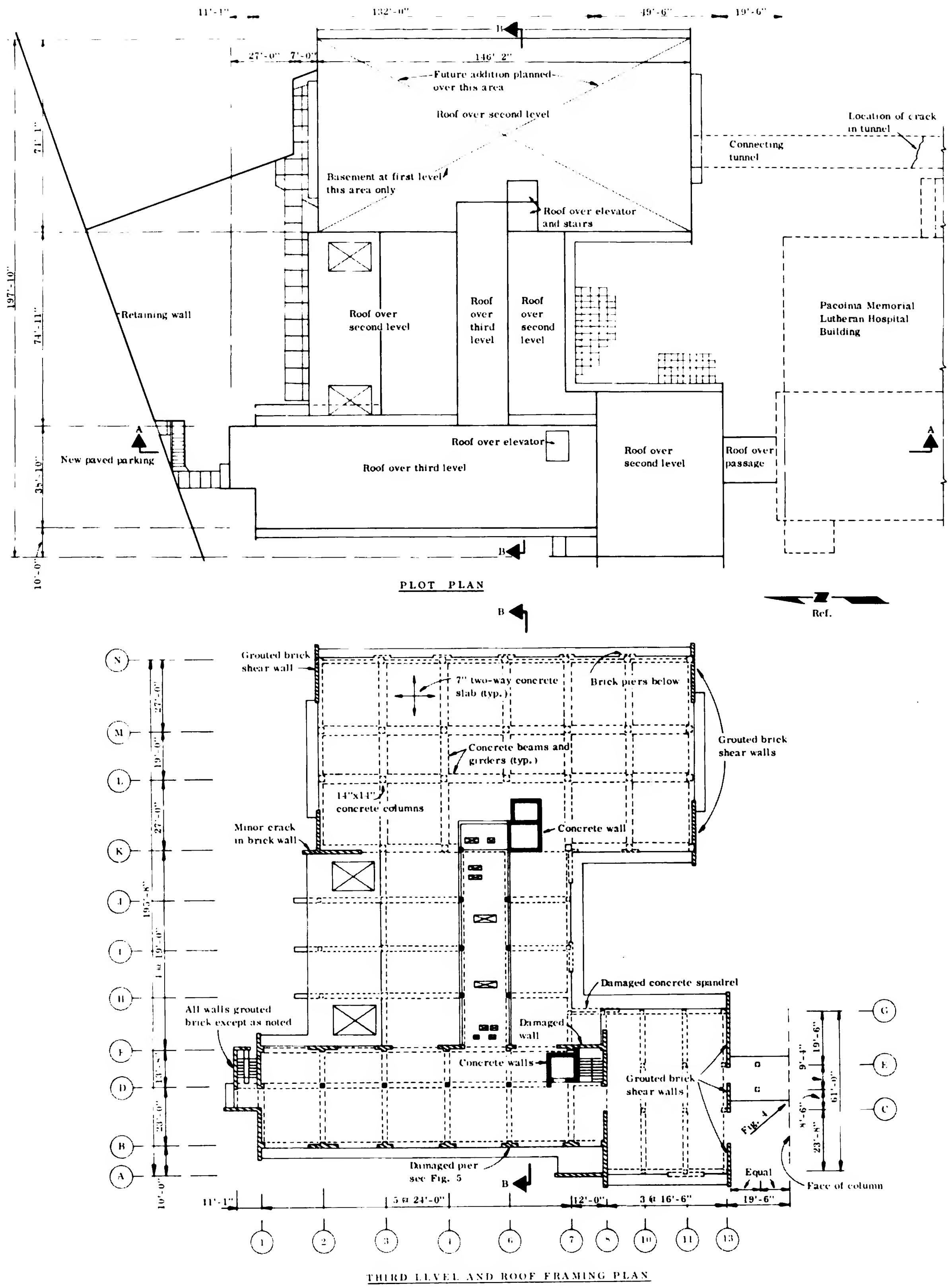

Figure 2.—Golden State Community Mental Health Center. Plot plan and third-level and roof framing plan.

Figure 3.—Golden State Community Mental Health Center. Immediately after earthquake, looking east. Thomas Wosser photograph.

Figure 4.—Golden State Community Mental Health Center. Canopy roof over passage to main hospital. Vertical offset at joint is 5½ inches. Paul Fratessa photograph.

have been caused chiefly by differential soil consolidation.

The most obvious damage occurred where the low roof over the passage abutted the main hospital building. Although there was no structural connection, the joint was flashed. Figure 4 shows a vertical offset of about 5½ inches, as well as some horizontal offset and evidence of pounding damage. The slab on grade immediately below also was offset vertically at the juncture of the two buildings. At line 13, where the canopy connects to the Mental Health Center, there was some minor spalling, but no evidence was found of flexural cracks in the canopy spandrels or slab.

Figure 5 is a photograph of the first-story (above-grade) brick pier at column line 6 in the west wall (line B). It will be noticed that the diagonal cracks appear in one direction only. The second-story pier immediately above this one also was cracked in the same direction. South is to the right in this figure.

Additional cracks and spalling were found in the line A brick wall near line 8 at the roof level; in the concrete spandrel on line G near line 7; and in the brick wall on the east side of the stairwell near line F–7.

In order to verify the concept of differential soil compaction as a principal cause of the damage, a series of elevations was taken on the ground level on a north-south line near line D, running the length of the west wing of the building. Results indicated a relative vertical movement of over 4½ inches between lines 1 and 13, with line 13 being the low end. A definite change in the rate of slope occurred near line 7. The pattern of deformation along this line is consistent with the fact that the principal damage observed occurred in the walls near column lines 6 and 7. The direction of cracking in the pier at line 6 is also consistent with a relative settlement of the south end. Figure 2, section A–A, shows that the piles at the south end of the Mental Health Center do not reach the approximate plane of denser soils, and, further, the upper portions of the southernmost of those piles may be located in an area that was backfilled during construction of the hospital. It is probable that the piles at the south end of the west wing did settle with respect to the rest of the building, and that this action was the major cause of the damage observed.

It was noted previously that the east wing contained a basement, and the piles in this area extended approximately 14 feet deeper than elsewhere. Relative movement was not observed in this area of the building.

The tunnel structure, which was not supported on piles, suffered a large crack through the roof and walls at the location shown in the plot plan (fig. 2). There was considerable differential movement within the tunnel. At the connection to the Mental Health Center, the tunnel had settled ½ inch, and from this point to the location of the crack there was a uniform slope down of 1½ inches. The tunnel connects to the hospital in two locations, where vertical offsets of 1½ and 3½ inches were observed. The filled area over the tunnel indicated that movement had occurred between the tunnel abutment and the

Figure 5.—Golden State Community Mental Health Center. Cracked grouted brick wall on line B at line 6, looking east. Thomas Wosser photograph.

hospital wall. The concrete walk over the tunnel was cracked on both sides of the tunnel, with the walk being at a higher elevation immediately over the tunnel and sagging downward in both directions going away from the tunnel. Again, there was clear evidence of earth consolidation.

SUMMARY AND RECOMMENDATIONS

The Golden State Community Mental Health Center performed very well. Its relative symmetry, good distribution of substantial shear wall elements, and general good detailing resulted in a building that is well tied together without any areas of major weakness. Because of its good performance, it is interesting to review the examples of good detailing as used in this building. These include:

Substantial trim bars at openings, edges of piers, etc.

Substantial (two No. 9 minimum) chord bars in slab all around exterior.

Column ties at all bars with closer spacing at ends and 12-inch maximum spacing.

Collector or drag bars to resisting elements.

Continuous top and bottom bars in beams and slabs.

Substantial doweling and tying in of grouted brick walls, etc.

The damage that resulted from differential earth movement is a further lesson that details and conditions perfectly adequate elsewhere may not be adequate in "earthquake country."

It is recommended that dynamic properties be considered in determining foundation criteria for an individual structure. In this building, it is probable that the differential structure settlement might have been eliminated or reduced by extending all piles to the same firmer strata under the structure.

Pacoima Memorial Lutheran Hospital (21)

11600 Eldridge Avenue, Pacoima

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GENERAL DESCRIPTION

The Pacomia Memorial Lutheran Hospital is the largest of three buildings in the Pacoima medical complex. It was dedicated in 1960 with a rather complex plan layout, as shown in the plot plan in figure 1. The hospital, which contains 110 beds, is a four-story concrete building, structurally separated into three units. A utility and service tunnel connects the central nursing unit with adjoining Golden State Community Mental Health Center.

The hospital is situated on a sloping site (fig. 1, section 1–1, and fig. 2). Because of the underlying unconsolidated alluvial materials, the foundations consist of drilled concrete piles, generally 20 inches in diameter and about 43 feet long. Support relies on skin friction with a single pile design capacity of 62 tons. Ground-floor slabs rest directly on soil and are not considered to contribute to pile loadings.

The foundation plan is shown in figure 3. While the superstructure is separated into three structural units, foundations are tied together into a single unit.

Pile caps are tied together with reinforced concrete tie beams, including diagonal members at the east and west ends of the central nursing unit. Lateral forces were considered to be transferred to the structure by pile bending and passive earth resistance on the pile caps and the diagonal tie beams.

Structural framing consists of cast-in-place, pan-joist concrete floors supported on concrete girders and columns. The one-way joist floors use a $4\frac{1}{2}$-inch-thick floor slab reinforced with 6- by 6-inch/No. 3 by No. 3 wire mesh, "draped" over joist steel and sagging to the bottom of the slab at midspan. Joists are typically $5\frac{1}{2}$ inches wide by $16\frac{1}{2}$ inches total depth at 3 feet on center spanning east and west. The typical framing plan is shown in figure 3. Typical girder, spandrel, and column details are shown in figure 4.

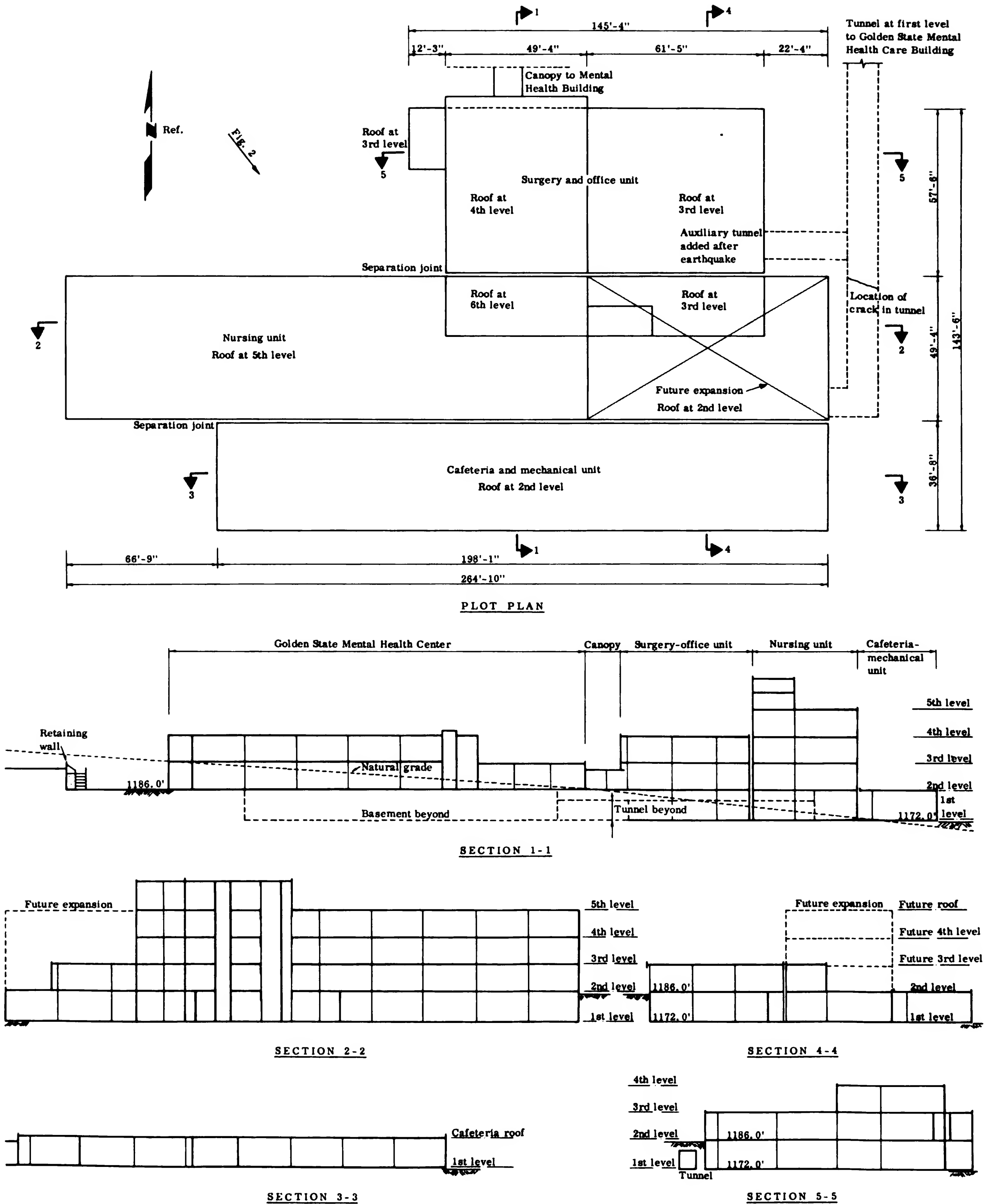

Figure 1.—Pacoima Memorial Lutheran Hospital. Plot plan and sections.

Figure 2.—Pacoima Memorial Lutheran Hospital. Immediately after earthquake, looking southeast. Ralph Goers photograph.

Concrete was stone concrete with a specified ultimate compressive strength of 3,750 psi at 28 days for walls, columns, beams, joists, slabs, and stairs with 2,500-pound concrete specified for all other locations. Intermediate-grade reinforcing steel was used. Typical reinforcing bar laps were 24 diameters except for column bars where 20-diameter laps were used.

CAFETERIA AND MECHANICAL UNIT

This unit, located at the south side of the building, is one story high with pan joists spanning to girders at 24-foot centers (fig. 5). The girders are supported on concrete columns on lines J and H, and cantilever 6 feet 8 inches toward the separation joint at line G. East-west (longitudinal) lateral forces are resisted by infill reinforced grouted concrete block walls on lines H and J (fig. 3). North-south (transverse) lateral loads are resisted by the grouted brick wall on line 1x and the two 8-inch concrete walls on the east end at lines 16x and 17. The concrete walls were reinforced with No. 3 bars at 11 inches on center each way in each face, except at line 17 which had No. 4 bars at 11 inches on center vertical on the inside face. Interior nonstructural concrete block walls near lines 10 and 11 were not assumed to resist lateral forces. Under these assumptions, the 36-foot 8-inch-deep roof diaphragm would span 168 feet for a 4.5 to 1 span-to-depth ratio. The grouted brick wall on line 1x is $9\frac{1}{2}$ inches thick, reinforced with No. 4 bars at 30 inches on center vertical, No. 5 bars at 30 inches on center horizontal, and is doweled to the floor slab with No. 7 bars at 12 inches on center, and to the grade beam with No. 4 bars at 30 inches on center.

NURSING UNIT

The nursing unit is the central and tallest portion of the hospital. At the time of the earthquake there were four levels for about two-thirds of the length of the structure, although provisions had been made for a future expansion that would continue the upper floors for the entire length of the unit (fig. 1).

Three different types of construction were used for the wall:

Nonstructural reinforced concrete block was used principally around the elevator shafts (fig. 3) and

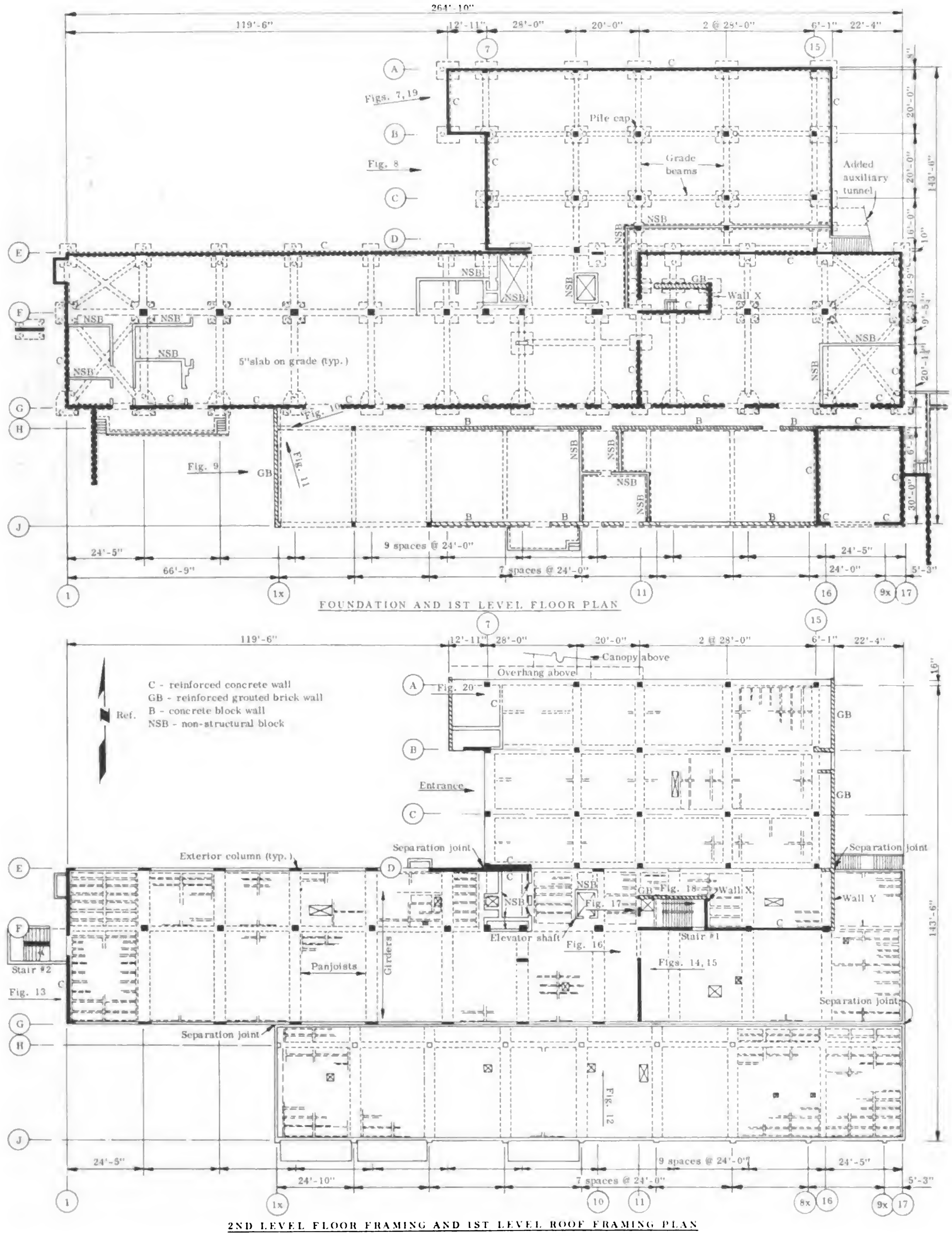

Figure 3.—Pacoima Memorial Lutheran Hospital. Foundation and first-level floor plan; second-level floor framing and first-level roof framing plans.

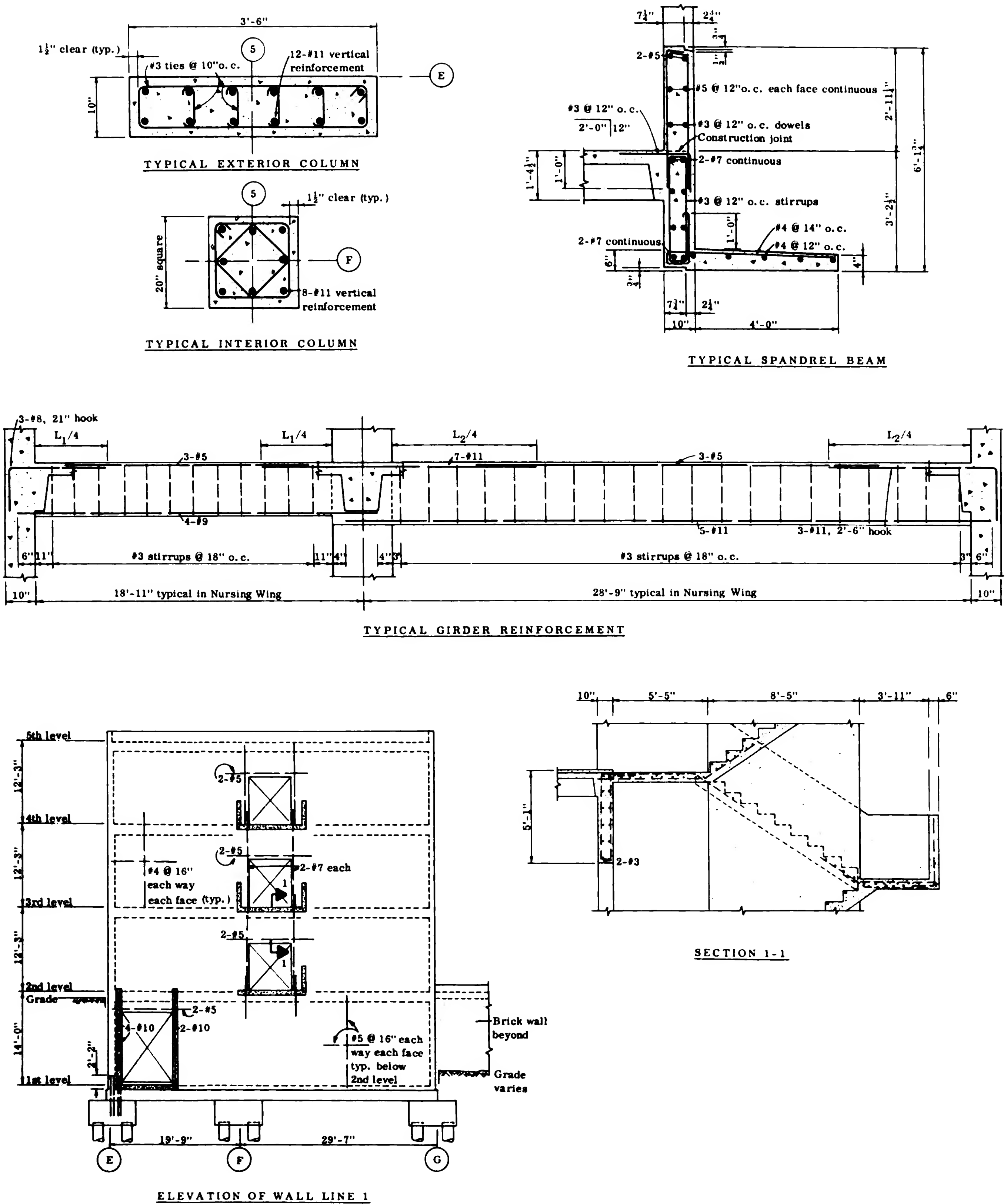

Figure 4.—Pacoima Memorial Lutheran Hospital. Typical exterior column, interior column, spandrel beam, and girder reinforcement; elevation of wall line 1 and section 1-1.

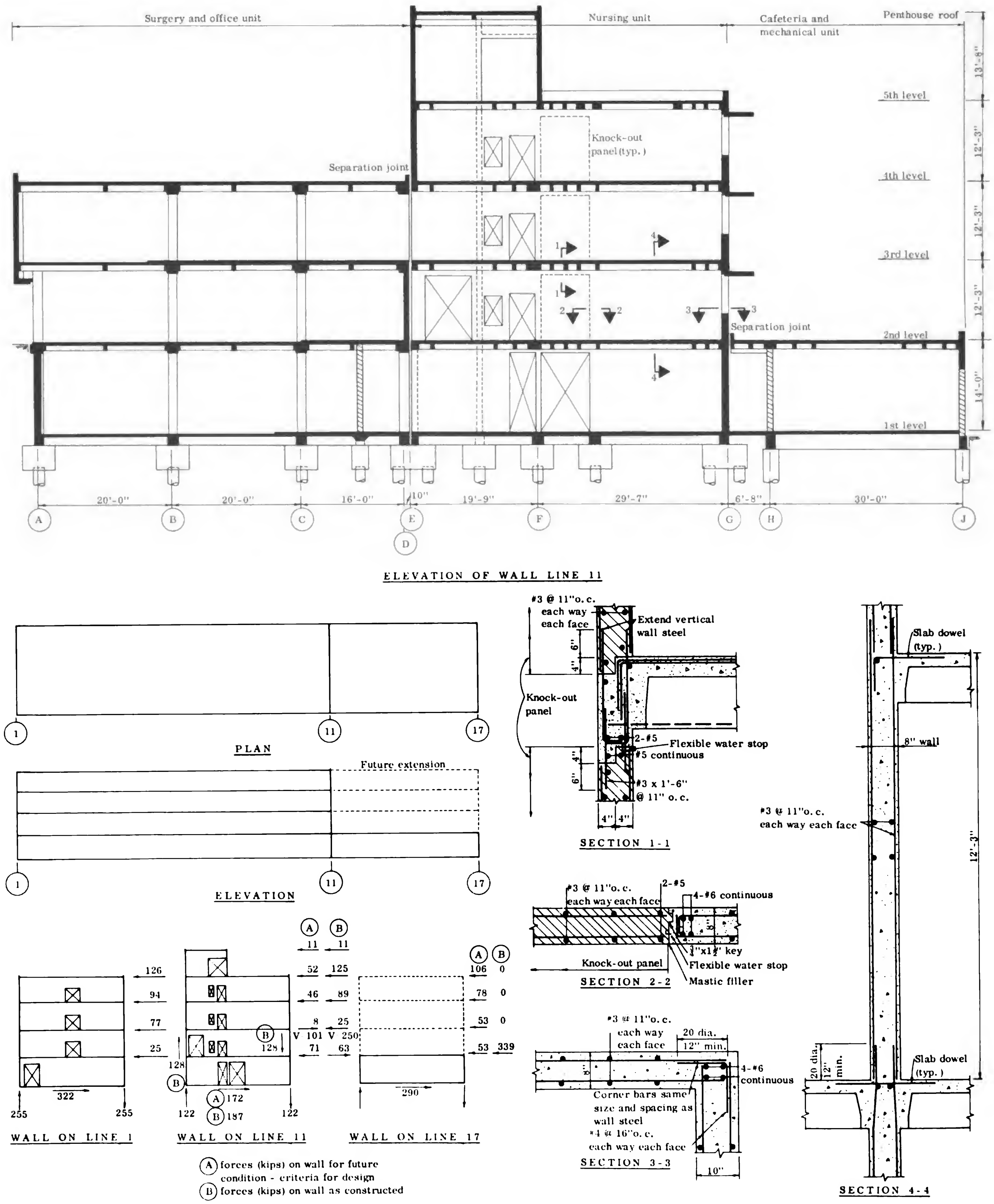

Figure 5.—Pacoima Memorial Lutheran Hospital. Elevation of wall line 11.

some corridors and rooms. These walls were not considered in the design to take lateral forces.

East-west lateral forces were resisted by a 9½-inch-thick grouted brick north wall of stair No. 1 (reinforced with No. 4 bars at 30 inches on center each way), and concrete walls on lines E, F, and G, plus the central concrete pier of stair No. 2. The exterior walls on lines E (as shown in fig. 2) and G are a spandrel-pier resisting system reinforced as shown in figure 4.

For north-south lateral forces, reinforced concrete walls on line 1 (10 in. thick, shown in fig. 4), line 11 (8 in. thick, shown in fig. 5), and the east wall of stair No. 1 (8 in. thick) were assumed to act. It will be noted that at the time of the earthquake, the portion of line 11 between F and G was an exterior wall, but contained three temporary reinforced concrete knockout panels that were to be removed when the future expansion addition was constructed. These panels were fitted in keys with mastic separations and did not have horizontal reinforcing dowels connecting knockout panels to the permanent shear-resisting concrete. The details at the boundary of these panels are shown in figure 5, sections 1–1 and 2–2.

SURGERY AND OFFICE UNIT

The most northern unit is the three-story surgery and office unit, with the typical vertical load framing. The lateral load-resisting elements are shear walls that vary from floor to floor in a rather complicated and unsymmetrical manner.

The north-south lateral forces are resisted on various levels as follows:

In the lowest (first-floor) level, forces are resisted by heavily reinforced 10-inch concrete walls, as shown in figure 3—one on line 7, one 12 feet 11 inches west of line 7, and the one long wall 6 feet 1 inch east of line 15.

In the next (second-floor) level, there are only two elements, both of reinforced grouted brick. One is a long wall 6 feet 1 inch east of line 15, while the other is the short wall 12 feet 11 inches west of line 7 (fig. 3). There was no shear key between this last wall and the concrete slab or foundations. Walls are 9½ inches thick, reinforced with No. 4 bars at 30 inches on center each way.

At the third level, there are concrete walls on lines 7 and 11 (fig. 6). The walls are 8 inches thick with

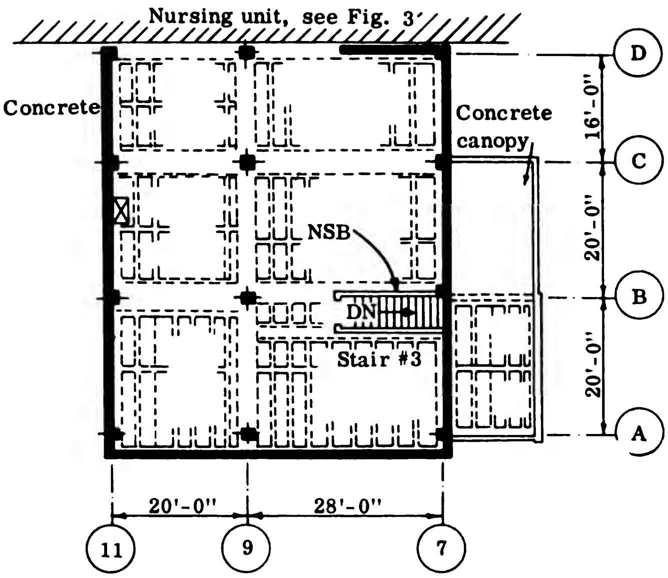

Figure 6.—Pacoima Memorial Lutheran Hospital. Partial third-level plan.

No. 3 bars at 11 inches on center each way in each face.

The east-west lateral forces are resisted on the various levels as follows:

On the lowest level, there are 10-inch heavily reinforced concrete walls on line A, a 13-foot-long wall on line B near line 7, and a 15½-foot-long 8-inch wall on line D near line 7 (fig. 3).

On the middle (second-floor) level, there is a 15½-foot-long 8-inch concrete wall on line D reinforced with No. 3 bars at 11 inches on center each way in each face, and a 9-foot-long 10-inch concrete wall on line B reinforced with No. 5 bars at 16 inches on center each way in each face. The small concrete block walls near column line B–7 were unable to take appreciable loads because of lack of dowels, lack of top and bottom restraint, and the unfavorable configuration.

On the third (top) level, there are two east-west elements—the 15½-foot concrete wall on line D (same as below) and the exterior 8-inch concrete wall that is 3 feet 5 inches north of line A (fig. 6). The concrete block walls at the stair near B–7 were assumed correctly to be unable to resist appreciable loads.

EARTHQUAKE DAMAGE

This hospital suffered major structural damage, to the extent that a substantial portion of the nursing

Figure 7.—Pacoima Memorial Lutheran Hospital. Vertical offset between catch basin and wall at brick wall 12 feet 11 inches west of line 7. Paul Fratessa photograph.

Figure 8.—Pacoima Memorial Lutheran Hospital. Slab on grade at main entrance. Paul Fratessa photograph.

unit and some parts of the other two wings have been demolished since the earthquake. The cost of repairs was estimated to exceed $1 million. The scope of this review was limited to information that was available before demolition.

Backfilled ground around the exterior walls settled up to 8 inches (fig. 7). At the main exterior entrance some of this ground settlement affected the floor slab on grade (fig. 8). As a result of this observation, levels were run on the second level to determine if the columns had moved vertically. It was concluded that there was no evidence of vertical settlement. Soil below the slabs on grade showed no signs of settlement when workmen cut through the slabs to install utilities after the earthquake.

An underground tunnel connects the nursing unit with the adjoining Golden State Community Mental Health Center (fig. 1). This suffered some settlement, which is mentioned in the discussion of the Health Center (Building Report 20).

DAMAGE TO CAFETERIA AND MECHANICAL UNIT

The only observed damage to this unit was the diagonal crack through the west brick wall (fig. 9). This crack extends through the concrete column, as shown in figure 10. There was evidence of pounding between the roof of this unit and the nursing wing second-level floor. Figure 11 shows the north end of the roof beam. The light substance at the expansion joint is Styrofoam joint material which has been forced out of the joint by compression. There was some damage to the flashing at the expansion joint. The concrete block walls and plaster partitions of this unit were virtually undamaged.

Using design code-required forces, the design unit shear stress in the west grouted brick wall would be 20 psi, resulting from a design reaction of 82 kips. The code at the time of design allowed 40-psi shear (with special inspection), as compared to the equivalent 1970 Uniform Building Code (section 24, table 24–H), which allows a maximum of 100 psi with special inspection.

It is probable that the crack in the wall resulted from extra loads caused by the nursing unit pounding against the roof slab of the cafeteria unit.

DAMAGE TO NURSING UNIT

The nonstructural concrete block walls at the ele-

Figure 9.—Pacoima Memorial Lutheran Hospital. Grouted brick wall at west end of cafeteria-mechanical unit. Note diagonal crack shown by arrow. Paul Fratessa photograph.

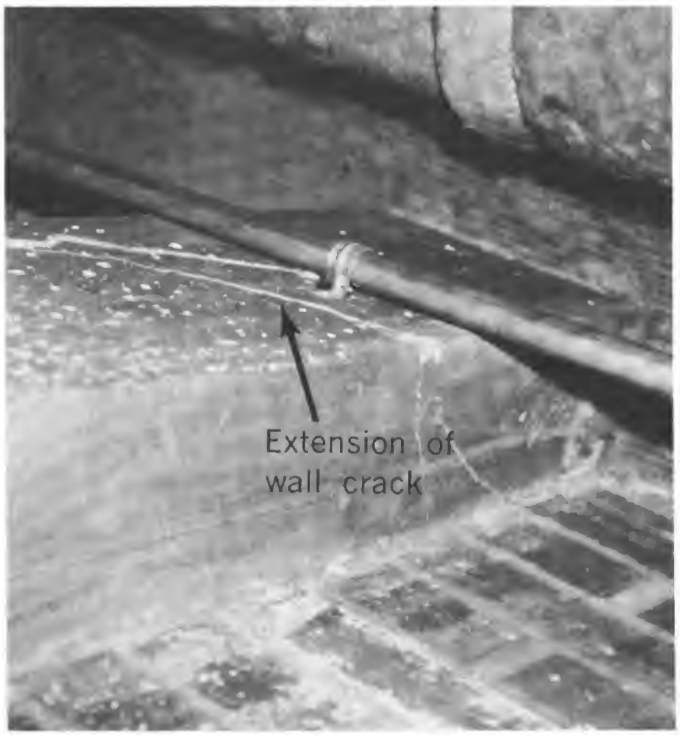

Figure 10.—Pacoima Memorial Lutheran Hospital. Concrete column at brick wall on line 1x at line H. Crack through column is an extension of wall crack shown in figure 9. Loring A. Wyllie, Jr. photograph.

Figure 11.—Pacoima Memorial Lutheran Hospital. Joint at roof slab and beam soffit between cafeteria-mechanical unit and nursing hospital at line 1x. Loring A. Wyllie, Jr. photograph.

vators at all levels and at the west end of the first floor were cracked and damaged extensively, with cracks up to 1/4 inch wide. Plaster partitions were damaged extensively at the lowest level, and to a lesser degree along the south wall at the upper levels.

Figure 12.—Pacoima Memorial Lutheran Hospital. Pier on line G at line 10 looking north. Loring A. Wyllie, Jr. photograph.

The south wall suffered concrete cracking at the second-level column, just west of line 11 as shown in figure 12. Other than this, no significant cracks were observed in this wall. No cracking or damage was observed on the north wall (fig. 2) except for several broken windows.

The exterior stair wall at the west end of the building was cracked slightly. Some spalling was evident at the connection of the wall to the landing.

The west wall of this unit (fig. 4, line 1) had diagonal cracks in both directions in both piers at the

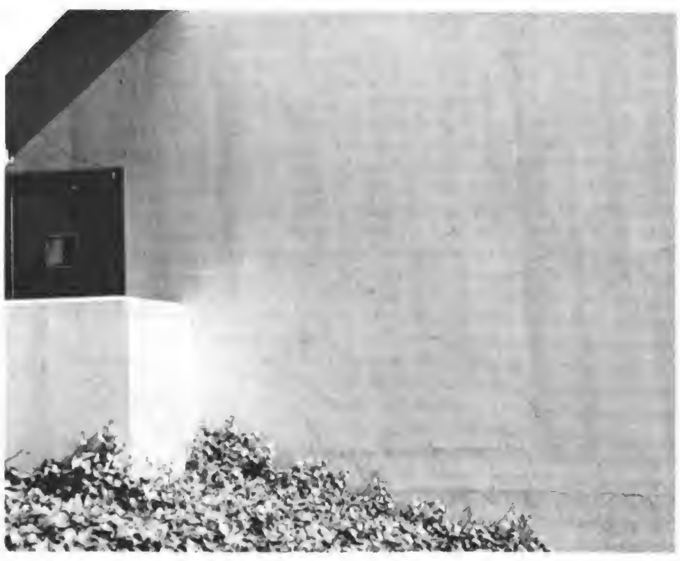

Figure 13.—Pacoima Memorial Lutheran Hospital. Shear wall at line 1, second level. Paul Fratessa photograph.

second level, but fewer similar cracks at the third level. Cracking was evident at the construction joints. The spandrel over the second-level door was cracked as shown in figure 15. Although it cannot be seen in figure 13, several of the diagonal cracks at the second level extend in a straight line through the third-floor construction joints and into the third-level wall. The wall below the second floor showed some diagonal cracks, which were smaller in number and size than those shown above the second floor.

The shear wall at the second level and above on line 11 (fig. 5) was cracked severely as shown in figures 14 and 15. The diagonal cracks of the wall extend through the construction joints at the floors. All construction joints showed movement. The knockout panels for future construction showed movement. The white patches seen in figure 14 evidently were made before the earthquake. The thin piers adjacent to the stairwell and to the north of the knockout panel were shattered at the second level, as shown in

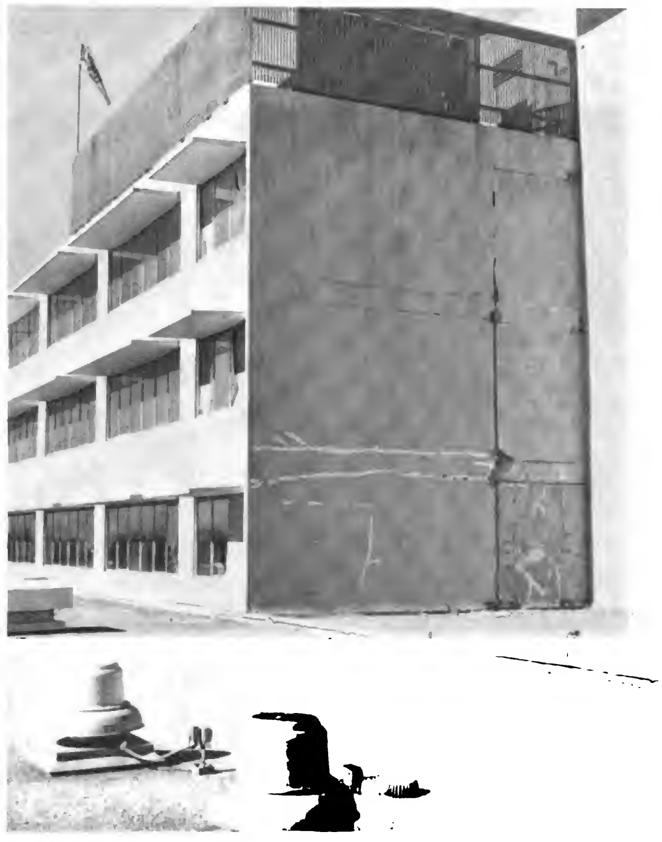

Figure 14.—Pacoima Memorial Lutheran Hospital. Elevation of shear wall on line 11. Patching of concrete prior to earthquake is visible. Note knockout panels at right side of wall. Paul Fratessa photograph.

figures 16 and 17. There was virtually no damage to this wall below the second floor.

The concrete stair wall at the east end of stair No. 1 (fig. 3) was cracked badly. The worst cracks were located at the horizontal construction joints shown in figure 18. Stair landing beams were damaged. The grouted brick north wall of this stair had several diagonal cracks at the second level. The south concrete walls were not damaged. The east-end wall on line 17 was undamaged.

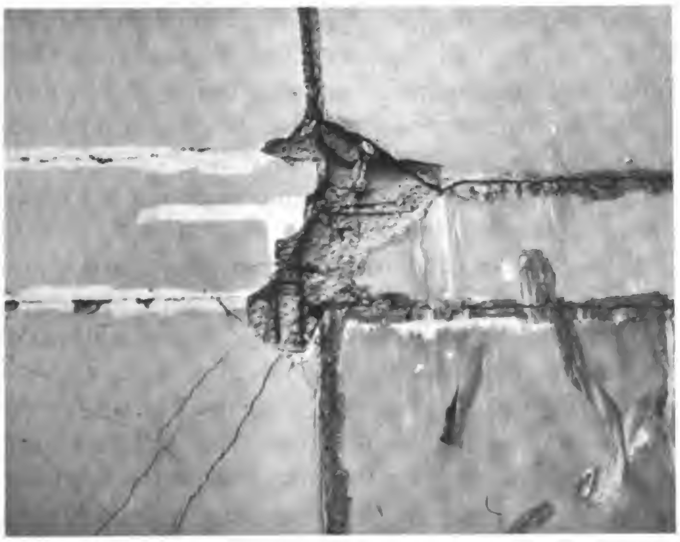

Figure 15.—Pacoima Memorial Lutheran Hospital. Closeup of damaged joint at knockout panel at third floor, line 11. Paul Fratessa photograph.

ANALYSIS AND DISCUSSION OF DAMAGE TO NURSING UNIT

The critical damaging motions on this building were those in the north-south direction, so the code design forces in this direction should be reviewed in detail. The design calculations were available and check calculations were made on the basis of the lateral code coefficient in use at that time; i. e., where $C = \frac{60}{4.5 + N}$ where C = coefficient percent of dead load and N = number of stories above the story under consideration. The calculations were quite involved because of torsional problems caused by varying rigidities and were further complicated by the planned future extension. The three major bracing elements on lines 1, 11, and 17 have been examined, omitting from the discussion the smaller elements noted as X and Y in figure 3 near stair No. 1.

Figure 16.—Pacoima Memorial Lutheran Hospital. Pier on line 11 at second floor, adjacent to large opening between lines E and F, looking east. Paul Fratessa photograph.

Figure 17.—Pacoima Memorial Lutheran Hospital. Thin pier on line 11 at second floor, adjacent to stair opening, looking east. Paul Fratessa photograph.

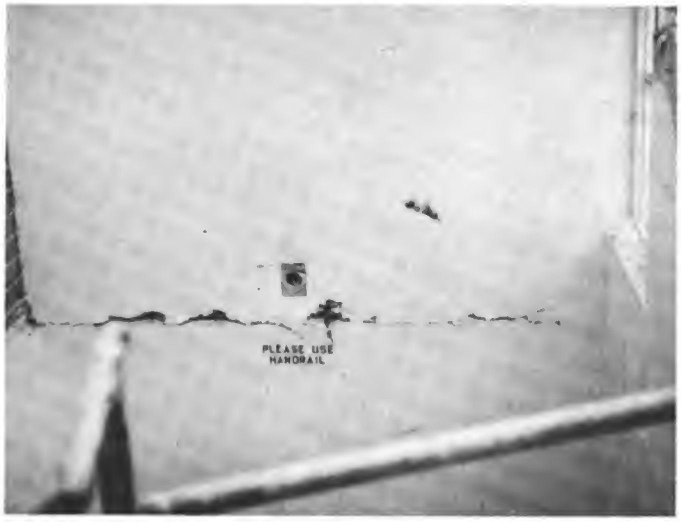

Figure 18.—Pacoima Memorial Lutheran Hospital. East wall of stair No. 1. Horizontal movement is at second-floor construction joint. Paul Fratessa photograph.

The design forces on the wall on line 1 (fig. 5) are shown as given in the design calculations. These forces were compared with the lateral forces required for future design and the forces for existing requirements, and no substantial differences were found. Under existing lateral loading conditions, the overturning moment would load the foundation piles to about 90 tons each. As noted before, no settlement occurred. The spandrel over the second-floor opening was cracked diagonally (fig. 13), but the spandrels above were not. It is possible that the pounding support of the cafeteria unit roof helped support this wall in one direction but not the other, which may help to explain the direction of the crack. On the whole, this wall performed very well.

At the other end of the building, where a future extension was planned, the design calculations for the wall on line 17 assumed the future condition as critical and provided a base shear of 290 kips. Because the wall at the time of the earthquake was only one story high, it could not take any diaphragm reactions at the third, fourth, and future roof levels. The forces that ordinarily would be delivered to it were carried by the wall on line 11 to the second floor, and a large portion was then transferred through the existing roof to the wall on line 17. This force, developed under code requirements, for the unexpanded structure would be somewhat higher than the original design force but would cause considerably less overturning. This probably explains why there was no damage to this wall.

The situation with the wall on line 11 is quite different. The building was designed for the future condition where all floor diaphragms would span from line 1 to line 17, with some support furnished at line 11 in accordance with its relative rigidity. Because of the many large wall openings, the wall on line 11 had a low rigidity, as compared to the walls on lines 1 and 17, and therefore would carry relatively small loads as shown in the (A) column (design forces) of wall line 11 in figure 5. In the condition that existed at the time of the earthquake, however, the future extension was not constructed, and there was no way to deliver loads to the wall on line 17. The full support of this portion of the building relied upon the strength of the wall on line 11. As shown at the wall on line 11 (fig. 5), the column (B) loads (code forces on wall as constructed) are substantially greater than column (A) loads above the second floor.

While very much attention was given in the lateral analysis to determining the loads on the walls as controlled by wall rigidities and the plan location of the center of rigidity, very little was done on the design of the walls to resist these loads. The ability of these three walls to resist those loads was based on the plan moment of inertia and shear area of the walls at various levels. No method was provided by which the overturning moment shears could be transferred through the thin shallow spandrels over the future openings on the walls on line 11 nor the existing openings. This subject is discussed in the earthquake report, *The Prince William Sound, Alaska, Earthquake of 1964 and Aftershocks,* U.S. Department of Commerce, ESSA, Volume II, Part A, pages 197–201, 1967. The method of relating overall resisting dead load moments to the calculated overturning moments, as tacitly approved by the code, does not highlight the internal forces within a pierced wall. As shown by photographs (figs. 14 and 15) and previous discussion, the resulting damage was substantial.

The action of the knockout panels is similar to that of infill walls. This action is different from a monolithic shear panel, in that shears are not transferred into the panel via the boundary interface, but principally are transferred in compression at the column-beam intersection. This creates an entirely different stress pattern on the boundary frame members, resulting in high shears and bending moments in the boundary frame near the intersection.

The damage pattern shown in photographs (figs. 14 and 15) clearly indicates this action, which is not specifically provided for in codes and was not anticipated in this structure.

DAMAGE TO SURGERY AND OFFICE UNIT

Neither structural nor architectural damage was observed at the lowest level of this unit. The solid concrete walls on the three sides, plus a portion of wall on the south side (fig. 3), were sufficient to resist the code-required forces at very low stress levels, and, evidently, deformations were sufficiently low to prevent architectural damage.

There was no structural damage and only minor, if any, architectural damage to the top (third) level of this unit. Again, the solid concrete walls on lines 11, 7, and Ax and the portion of wall on line D (fig. 6) were sufficient to resist the forces at reasonable deformations. Note that this statement can be made in spite of the fact that the north wall (on line Ax) is cantilevered 3 feet 5 inches north of the support line below. The ends of this wall are supported in turn by solid concrete walls extending past line A to resist the overturning forces adequately.

The second level, where the main entrance is located, was damaged very severely. The floor slab at the entrance (near B–7) was cracked (fig. 8). The grouted brick shear wall between lines A and B, 12 feet 11 inches west of line 7, shattered along the base and moved about ½ inch to the south (fig. 19). The concrete wall north of the entrance (line B near line 7) suffered small diagonal cracks throughout. The concrete wall on line D at the west end of the unit was not damaged. The interior concrete block walls at the stairs and electrical closet near lines B and 7 were shattered badly. When these walls were removed, it was found that there were no dowels from slab to block wall as are customarily used. The concrete column at A–7 (fig. 20) and the adjoining two columns to the east were cracked. Evidently, this was due to north-south motion. The concrete at the column at C–7 was spalled badly. No damage was observed at other columns. The grouted brick shear wall 6 feet 1 inch east of line 15 was not damaged.

It is evident that the rigid unit above the second level rotated about the more rigid east grouted-brick shear wall. There was insufficient strength in the three small shear walls near line 7 to resist the shear and torsional forces. The grouted brick wall at the west end of the unit near the entrance would have been stressed to 26 psi in shear under the code-required force of 63 kips. As previously mentioned, the code at the time of design would allow a maximum of 40

Figure 19.—Pacoima Memorial Lutheran Hospital. Grouted brick wall 12 feet 11 inches west of line 7, looking east. J. F. Meehan photograph.

Figure 20.—Pacoima Memorial Lutheran Hospital. Column A–7 above second level; cracking is in plaster. Cracks in concrete were in same pattern. Paul Fratessa photograph.

psi, while the 1970 UBC would allow a maximum of 100 psi. Although it had only minimum reinforcing, there were no signs of diagonal cracking. However, since there were no shear keys from the brick to concrete the wall moved as a unit on the concrete. The only mechanical connection was the No. 4 dowels at 30 inches on center to resist a shear of 3,000 pounds per lineal foot. Once this most rigid element failed, the western end of the building could move north and south and cause secondary cracking of the columns, concrete block walls, and the interior partitions.

REPAIRS

It was estimated that repairs to the hospital would have cost over $1 million. The nursing unit was demolished to the second level, while the other two units are being repaired and remodeled for temporary use.

SUMMARY

The Pacoima Memorial Lutheran Hospital is a building with a complex layout consisting of three structurally separate units. Of these units, the tallest and central unit was damaged severely, primarily due to the poor performance of the eastern shear wall above the second floor. This performance was affected by nonstructural knockout panels as well as by often-overlooked overturning forces through spandrels. The three-story surgical unit was damaged at the second level due to high torsional forces resulting from the sliding of the west shear wall. The one-story cafeteria unit had little damage except to the west wall. This damage most likely can be attributed to pounding of the four-story central unit.

COMMENTS AND RECOMMENDATIONS

It is evident that this structure was subjected to very severe ground motion. The performance of the cafeteria and mechanical unit and the first and third levels of the surgery and office unit was excellent. The drilled pile foundations performed well, even though quite heavily stressed. The performance of the wall on line 1 of the nursing unit is quite reassuring.

The extensive damage to the second level of the surgery unit and the major portion of the nursing unit, however, suggests that improvements are necessary in code requirements, construction inspection, and design practices. In this structure, the newer (1970) code forces would be about the same as those required for the design of this building and allowable design stresses in shear walls would have been lower.

Specific recommendations resulting from the performance of this structure are as follows:

1 More attention should be paid to compaction requirements and to inspection of backfill of soil outside of and adjacent to the structure, to reduce differential settlement between the structure and surrounding soil.

2 In designing structures with future additions, the interim stress path must be examined more closely to determine the critical condition.

3 The "notch effect," whereby relatively weak elements such as the second floor of the surgery unit are adjoined by rigid elements (first and third levels), must be given more consideration.

4 The architectural layouts must permit the structure to be simplified, and more and stronger resisting elements should be permitted than, as an example, was done at the second level of the surgery unit.

5 Codes must require an adequate analysis of the internal elements of shear walls. The "vertical shear" caused by overturning forces through spandrels must be analyzed and provided for.

6 Construction inspection is required to assure that dowels, keys, and reinforcing are provided where required by the design drawings.

7 Additional stirrups and ties are needed at the ends of columns and girders where infill walls or knockout panels cause high shear stresses.

8 Construction joints have been a recurring point of weakness in all earthquakes. Keys (and possibly diagonal reinforcement) to carry the entire shear force must be provided.

Indian Hills Medical Center (22)

14935 Rinaldi Street, Los Angeles

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Photographs provided by Portland Cement Association.

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GENERAL DESCRIPTION

The Indian Hills Medical Center is on the northeast corner of Rinaldi Street and Indian Hills Road. It faces south on Rinaldi. It is seven stories high with a small mezzanine floor near the core area for equipment (figs. 1, 2, and 3). About 600 feet north of this building is the one-story convalescent home, a unit of the Holy Cross Hospital. About 300 feet to

Figure 1.—Indian Hills Medical Center. Northwest corner.

Figure 2.—Indian Hills Medical Center. Southeast corner.

the northwest is the seven-story Holy Cross Hospital main building. Northwest of this main building is a one-story service building. The buildings of the Holy Cross complex are described in Building Report 23.

The medical center building is a reinforced concrete structure with a complete vertical load-carrying frame (figs. 4 through 8). It was designed under the 1966 edition of the Los Angeles City Building Code using K = 1.0. (See page 26 for explanation of K factor.) Concrete beams run north and south across the building. The typical floor slabs spanning 19 feet between girders are 6½ inches thick. The exterior walls are of light curtain wall construction, except where the concrete shear walls are located.

Interior partitions are metal stud and plaster and/or gypsum wallboard. The shear walls are 8 inches thick with steel reinforcement in the center. The typical shear wall reinforcement is No. 5 bars at 18 inches on center each way.

Portions of the shear walls below the floor slabs are reinforced as beams capable of carrying the adjoining floor and walls (fig. 9, section 4–4). At the north and south walls, where the slabs span parallel to the walls and where there are no shear walls, the slab is not thickened, but additional steel is added in the slabs (fig. 9, section 7–7).

The mezzanine floor is hung from the second floor by solid wall beams spanning north and south along lines 3, 4, 7, and 8 (figs. 5 and 6).

There is no basement (fig. 4). The ground floor slab is 4 inches thick, reinforced with 6- by 6-inch/No. 10 by No. 10 electrically welded mesh. A 4-inch gravel base is provided under this concrete slab.

The foundation materials vary from sandy clays, silty sands, sands, and gravelly sands, in this order below the surface, according to the soils report. Water was not encountered in the 50-foot test bor-

Figure 3.—Indian Hills Medical Center. Southwest corner.

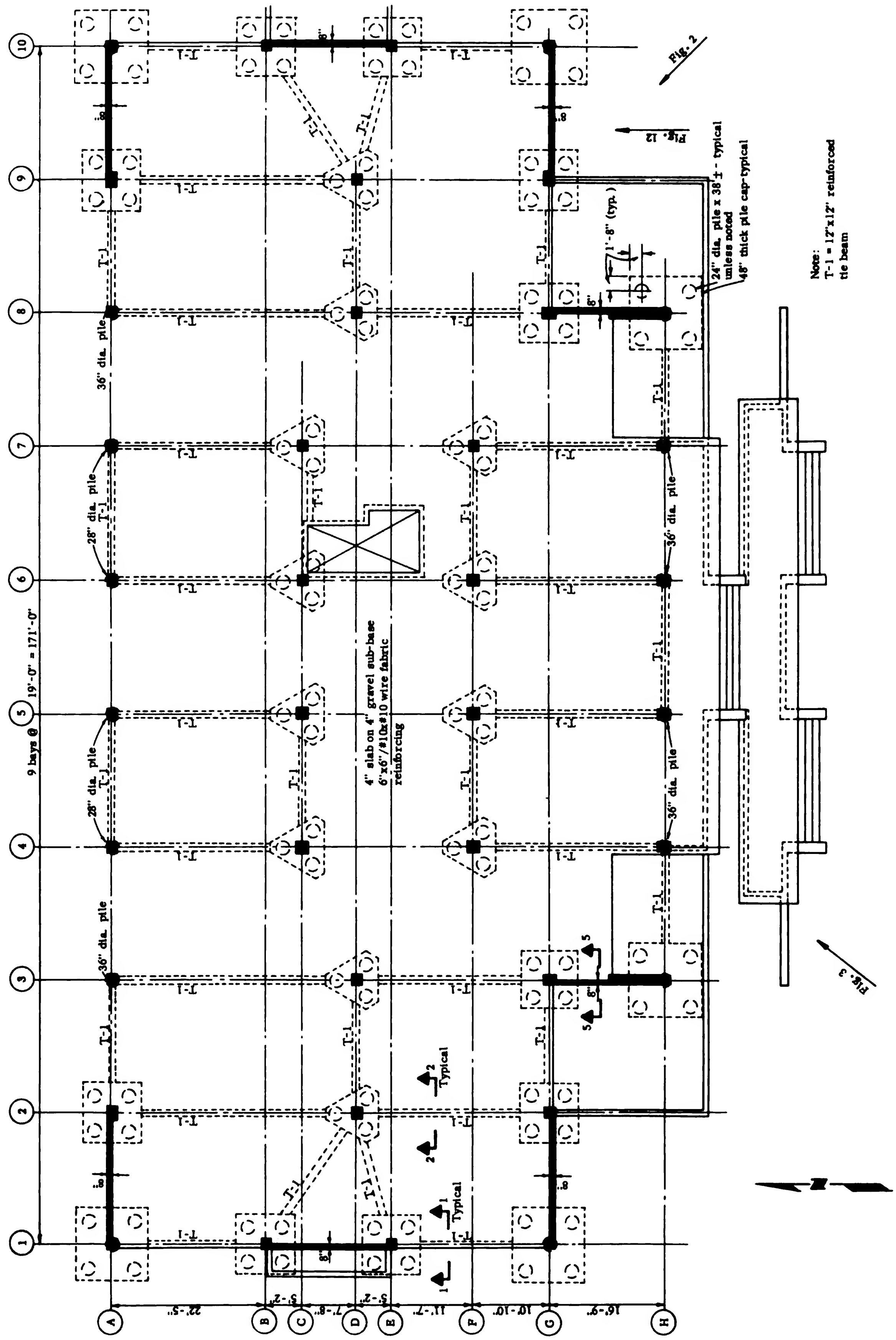


Figure 4.—Indian Hills Medical Center. First-floor and foundation plan.

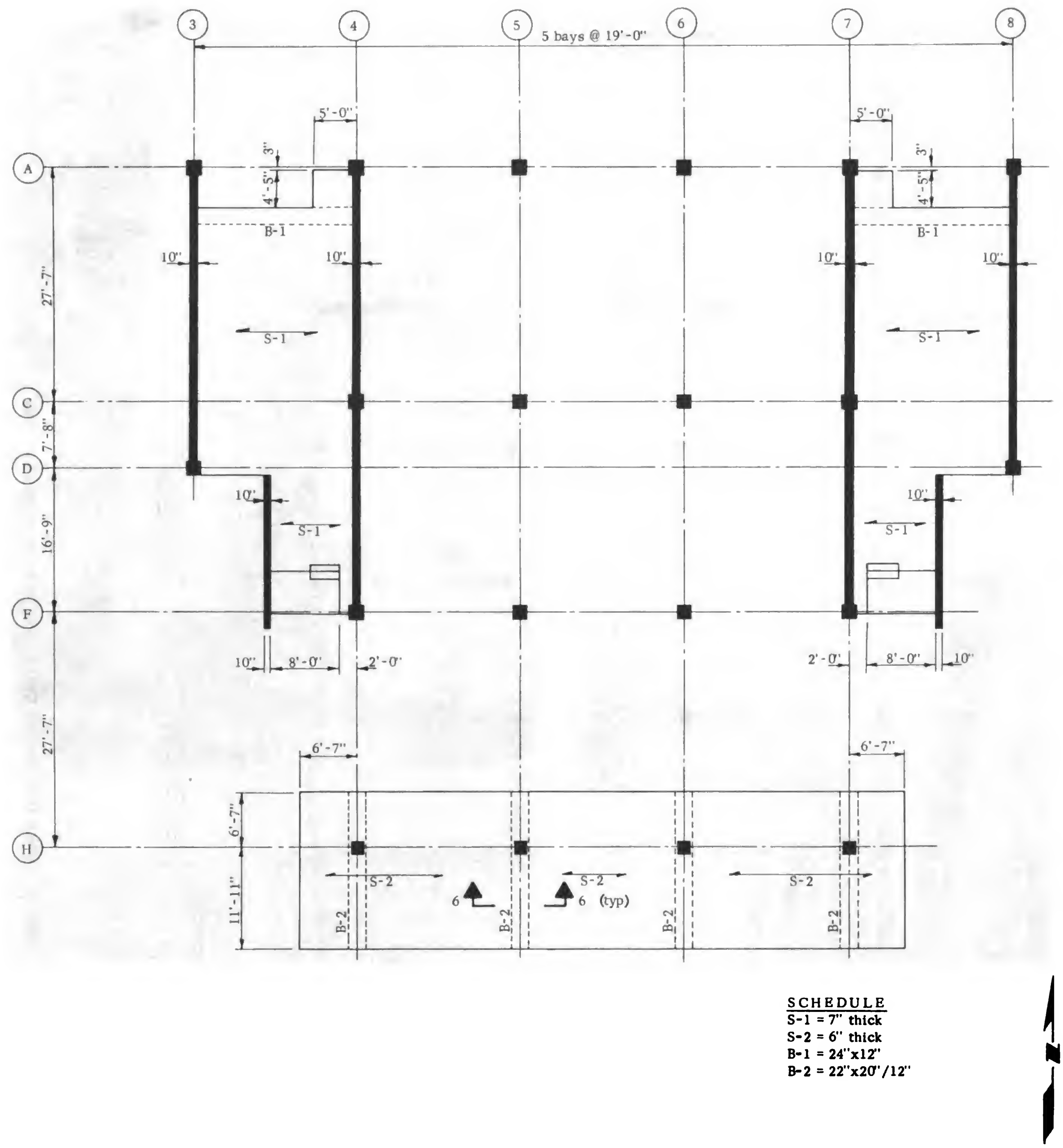


Figure 5.—Indian Hills Medical Center. Mezzanine-floor framing plan.

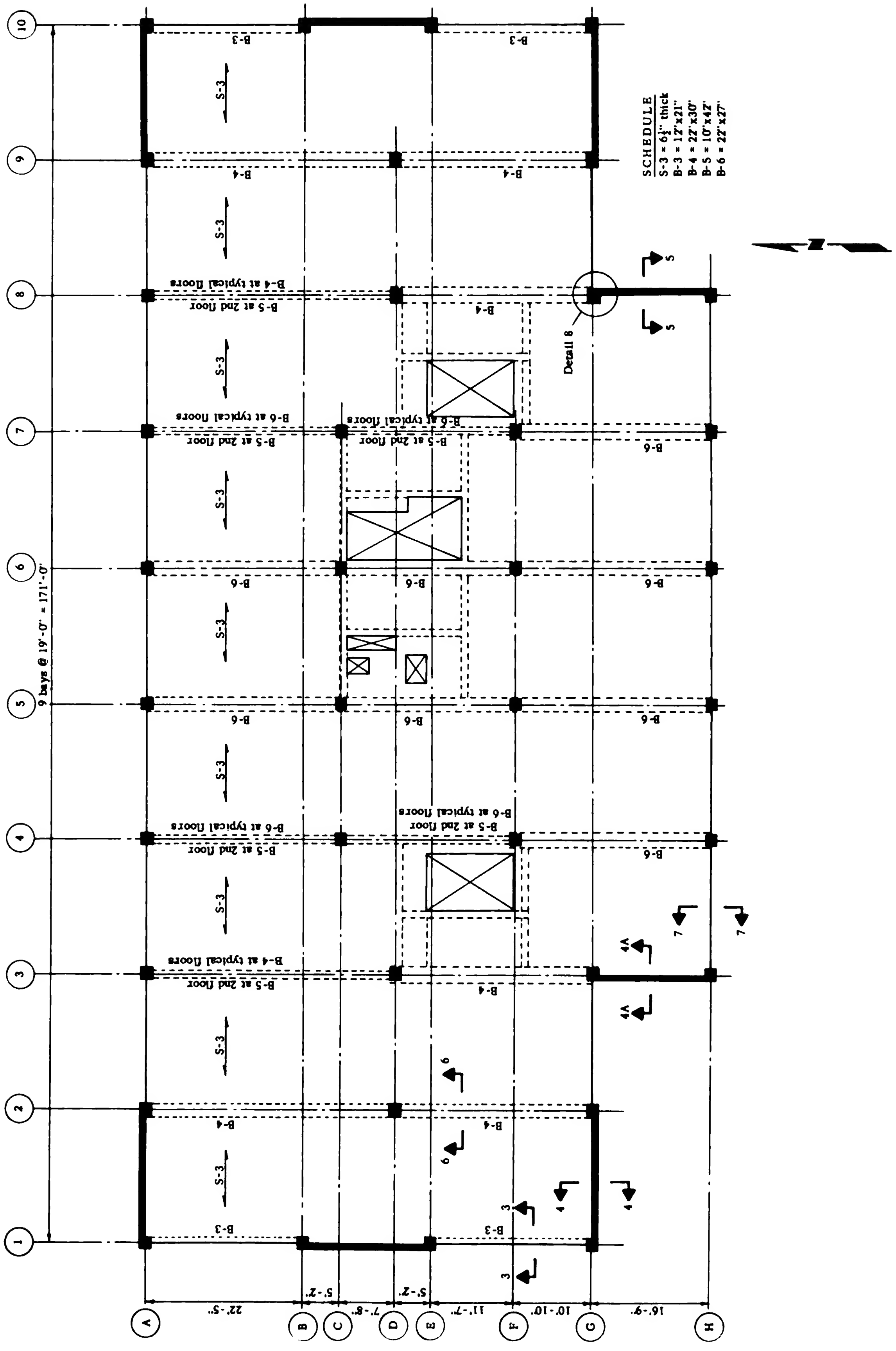


Figure 6.—Indian Hills Medical Center. Second-floor framing plan. Third, fourth, fifth, and sixth floors are similar.

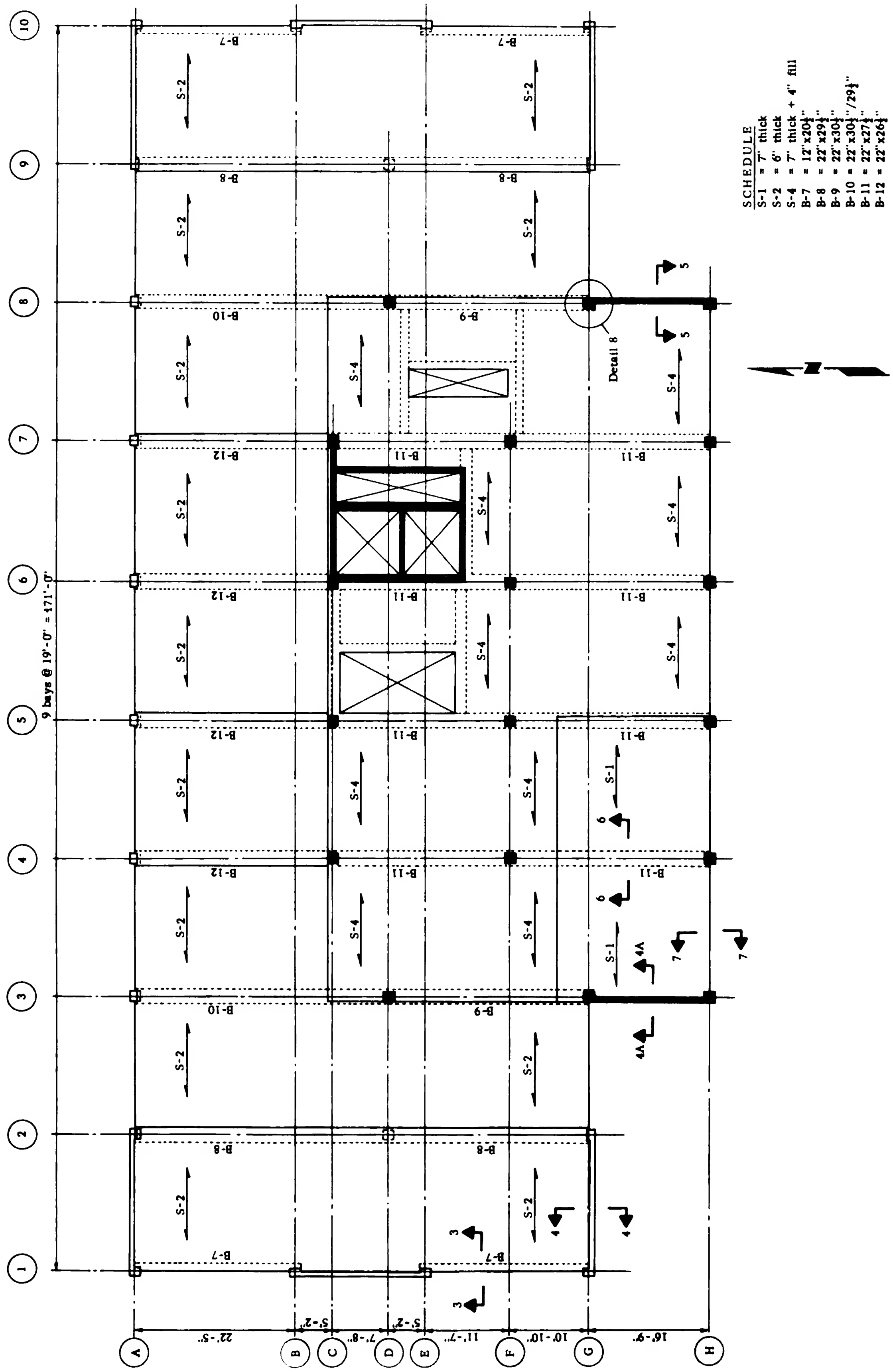
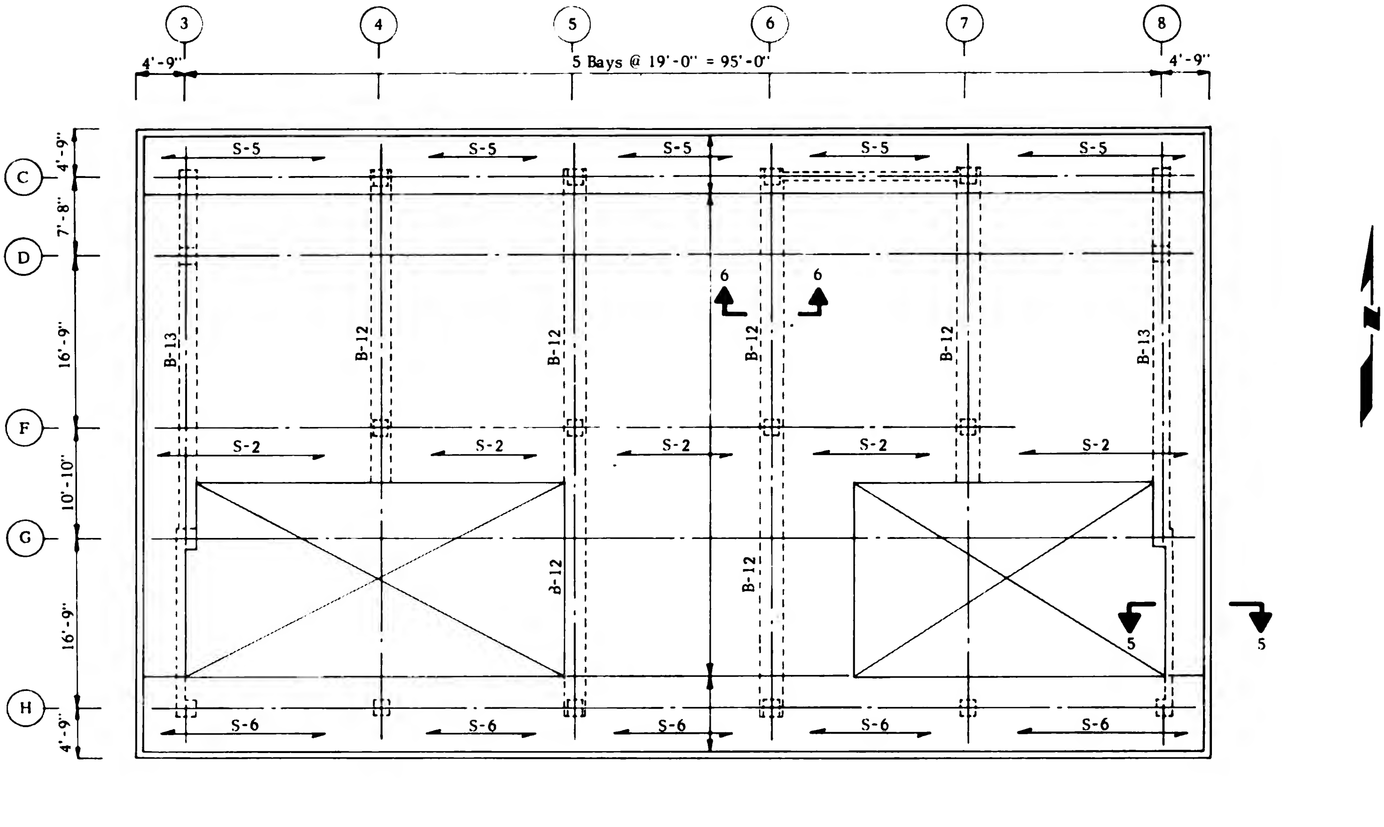


Figure 7.—Indian Hills Medical Center. Roof and penthouse floor framing plan.



SCHEDULE
S-2 = 6" thick
S-5 = 9" thick
S-6 = 12" thick
B-12= 22"x26½"
B-13= 19"x29½"

Figure 8.—Indian Hills Medical Center. Penthouse roof framing plan.

ings. Drilled and cast-in-place concrete piles were used to support the building (fig. 4). Pile lengths vary from 35 to 40 feet below grade. Piles are reinforced with vertical steel and ties. Pile diameters vary from 24 to 36 inches. Pile caps are interconnected with 12- by 12-inch tie beams (fig. 9, section 1–1).

Concrete for structural floor and roof slabs and beams is lightweight concrete (115 pcf). All other concrete is rock concrete. The lightweight concrete at floors extends into the stone concrete shear walls and columns.

The specified strength for slabs and beams, second floor through roof (figs. 6 through 8), is 3,000 psi. The specified strength for columns and walls below the second floor and the canopy slabs and beams is 5,000 psi. Columns and walls above the second floor and the mezzanine slab are specified to be 3,750 psi concrete. Slabs on grade are specified as 2,000 psi concrete, and piles and pile caps as 2,500 psi concrete.

Reinforcing steel was specified to be ASTM A–431 for longitudinal reinforcement in the lower columns, and A–432 in the upper columns and for main reinforcement in beams and slabs. A–15, intermediate grade, was specified for all other reinforcement. Typical bar laps of 30 diameters are required.

Column bars were lap-spliced. Column ties consisted of No. 2 ties at 6 inches on center for columns at the ends of shear walls, and for other columns No. 2 ties at 12 inches on center where longitudinal reinforcing is No. 9 or less, and No. 3 ties at 18 inches on center for larger bars.

In general, the structural system is regular and symmetrical. It is a structure that can be modeled easily to predict elastic response to earthquakes.

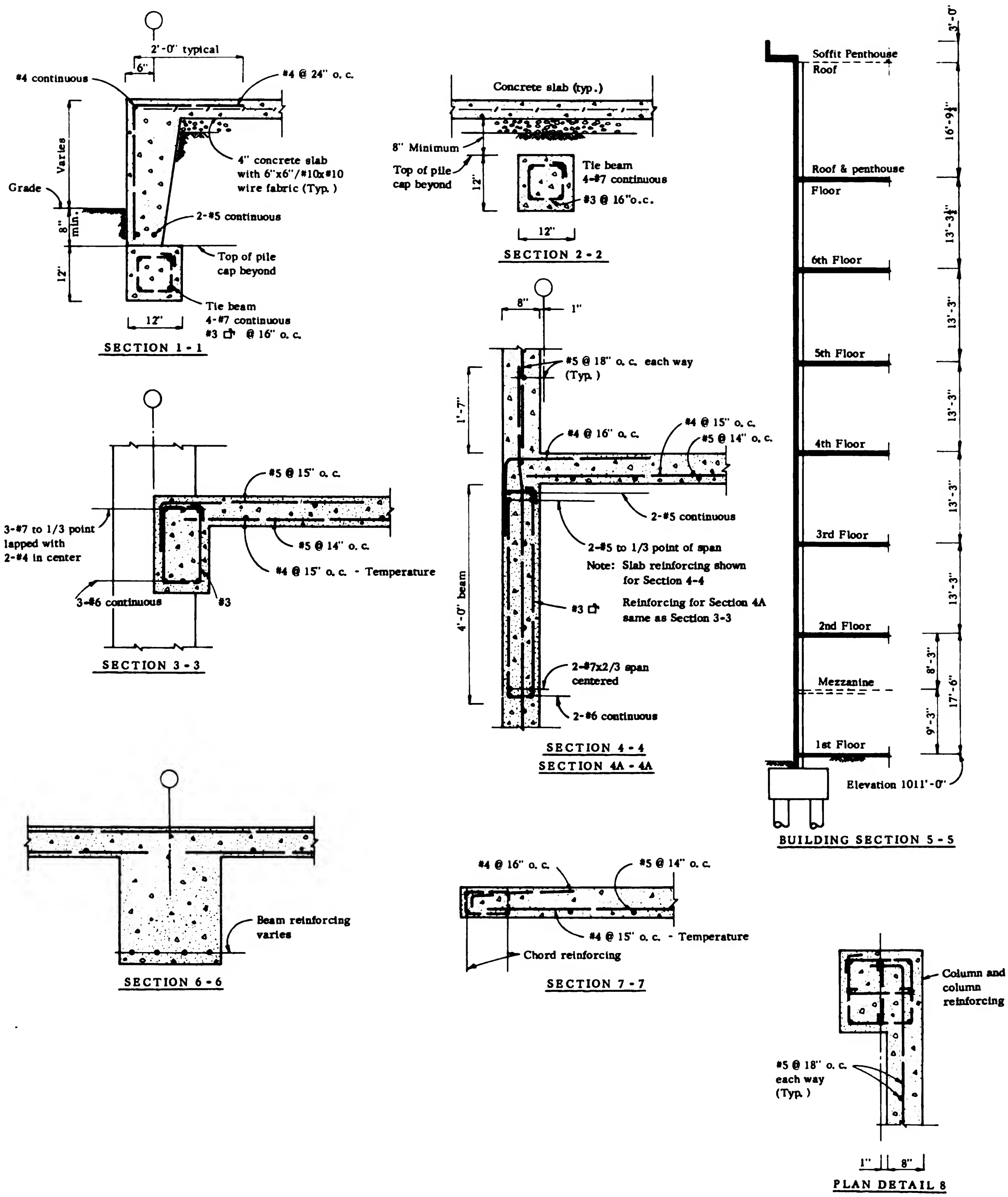


Figure 9.—Indian Hills Medical Center. Sections and detail.

Figure 10.—Indian Hills Medical Center. North-side shear wall, east end.

Figure 12.—Indian Hills Medical Center. South-side shear wall, east end.

EARTHQUAKE DAMAGE

Some of the shear walls were damaged by cracks at the horizontal construction joints at floorlines (figs. 10 and 11). There is evidence of slight horizontal movement. These cracks are not related directly to shear as a measure of diagonal tension. The slab depth is 6½ inches thick, and, in some cases, cracks occurred at the top and bottom of this intrusion of 3,000 psi lightweight concrete into the higher strength rock concrete of the shear walls proper.

The ends of the shear walls were designed as columns and were subjected to shear and axial load stresses. Concrete in the splice area crumbled (fig. 11). All of the shear walls in the lower levels cracked in the typical **X**-pattern (figs. 10 through 14), indicating high shear stresses.

Figure 11.—Indian Hills Medical Center. North-side shear wall, west end.

Figure 13.—Indian Hills Medical Center. Shear wall X-cracking, third floor.

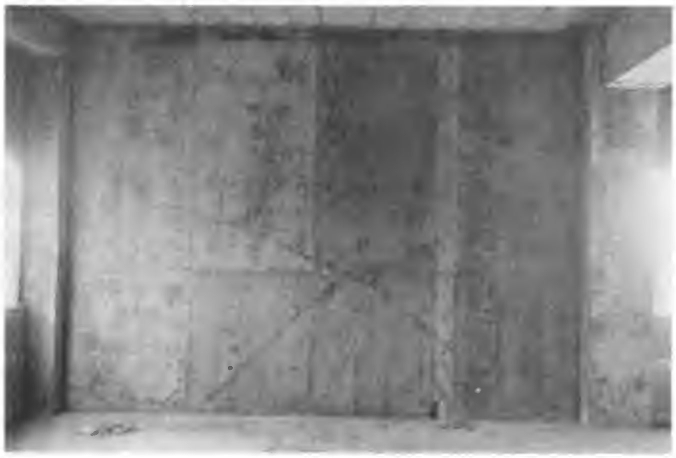


Figure 14.—Indian Hills Medical Center. Shear wall X-cracking, fourth floor.

Figure 15.—Indian Hills Medical Center. Third-floor exterior column damage.

In several cases where shear walls were tied into the building proper by concrete girders having a significant moment of inertia, damage was noted at the intersection of girder and wall (figs. 15 and 16). The girders run in a north-south direction. Damage was noted where considerable restraint in bending was offered by the girders (fig. 17). However, in at least one case, damage was found in an interior column-girder connection (fig. 18). The 6½-inch-thick floor slab usually is considered a rather stiff diaphragm. Some yield or inelastic deformation must have occurred, either in the diaphragm and/or the walls, to allow the interior beam-column connection to become damaged. The distribution of this type of damage indicates that the failure of the splicing of shear wall end reinforcing was the principal reason for this inelastic deformation, even though there was some cracking in the floor slabs.

No evidence was found that differential settlement in the building contributed to damage.

Figure 16.—Indian Hills Medical Center. Fourth-floor exterior column damage.

Figure 17.—Indian Hills Medical Center. Fourth-floor exterior girder and column damage.

The typical **X**-cracking in shear walls was in the east and west shear walls of the north elevation below the fifth floor, as well as in the center shear walls of the east and west elevations below the fifth floor. On the south elevation, diagonal **X**-cracks occurred only below the fourth floor. The **X**-cracking produced fairly uniform crack widths in both north-south and east-west walls. The location of this more pronounced **X**-cracking in the lower floors indicates that the building accelerations were, in general, in the first mode.

In several locations, the walls offset transversely up to a maximum of 3/4 inch at the floor pour line, indicating an offset of the reinforcing at these points (fig. 1). At the first floor on the eastern edge of the southeast shear wall facing Rinaldi Street, the wall was offset severely, indicating the chord reinforcing at this location had become offset (fig. 12).

A more detailed description of damage can be taken from the repair drawings. The tabulations of damage details in table 1 is from this source. The location symbols refer to the grid lines. The repair specifications are not included in this report. In general, repair consisted of adding new gunite walls on the exterior of original walls, removing and replacing spalled areas, weld splicing new reinforcing segments where bars were offset by the earthquake, and repairing cracks by epoxy. The cost of these repairs came to $150,000, which is approximately 9 percent of the original cost of the building.

There was no evidence of permanent ground displacements or damage in the area of this structure.

Figure 18.—Indian Hills Medical Center. Third-floor interior girder and column damage.

Table 1.—Description of damage at specific locations

Floor	Location	Damage description
2	A8	Top connection, hairline cracks, beam sides.
	H6	Top connection, minor hairline crack.
3	A6	Top connection, spalling, column under beam.
	A7	Top connection, hairline crack, beam side; spalling.
	A8	Top connection, hairline crack, beam side; bottom column spalling.
	A9	Top connection, hairline crack, beam side; spalling.
	D9	Top connection, column spalling.
	B10	Top connection, hairline crack.
	E10	Do.
	H6	Top connection and bottom of column, hairline crack.
	H7	Top connection, hairline crack.
	G9	Bottom column, major spall.
4	A2	Top beam side, hairline crack, and bottom column spall.
	A3	Top beam side, hairline crack.
	A4	Do.
	A6	Top connection, hairline crack on beam; spall, top of column.
	A7	Do.
	A8	Do.
	A9	Top connection, hairline crack on beam; spall, bottom of column.
	B1	Top connection and bottom of column, spalling.
	E1	Top connection, spalling.
	D2	Top connection, hairline crack on beam side.
	C6	Top connection, hairline crack on column.
	F7	Top connection, hairline crack on beam.
	D8	Top connection, beam and column, hairline crack.
	D9	Top connection at column, spall.
	E10	Top and bottom of column, spall.
	G3	Top connection at column, hairline crack.
	H4	Top connection, hairline crack.
	H5	Do.
	H6	Do.
	H7	Top connection at column, spall; beam side, hairline crack.
	G8	Do.
	H8	Bottom column, crack.
	G9	Top connection, hairline crack.
	G10	Top connection, spall.
5	A2	Top connection, hairline crack.
	A3	Top connection, beam and column, spall.
	A4	Top connection of beam, hairline crack; top of column, spall.
	A5	Do.
	A6	Do.
	A7	Top connection of column below beam, spall.
	A8	Top connection, small spall.
	A9	Top connection, hairline crack and spall.
	A10	Top connection, spall.
	E1	Top of column, crack.
	D9	Top connection, hairline crack.
	B10	Top connection, spall.
	E10	Do.
	G2	Top of column, hairline crack.
	G3	Top of beam, hairline crack; top of column, spall.
	H4	Do.
	H5	Do.
	H6	Top of beam, hairline crack.
	H7	Top connection, spall.
	G8	Top connection, hairline crack and spall.
	G9	Top connection, crack.
	G10	Top connection, cracks and spalling.
6	A3	Bottom of column, spall.
	A4	Do.
	A6	Do.
	A7	Top connection, cracks; bottom of column, spall.
	A8	Top connection and bottom of column, cracks.
	A9	Top connection, cracks.
	E1	Top connection, column spall.
	D8	Top of column, hairline crack.
	D9	Top of column, spall.
	E10	Top connection, cracks.
	G2	Top connection, cracks, beam and column.
	G3	Top connection, cracks and spalling.
	H4	Bottom of column, cracks.
	H5	Top of column, spall.
	H6	Top connection, spall; bottom of column, crack.
	H7	Top connection and bottom of column, cracks.
	G8	Top connection, spall.
	B10	Do.
	G9	Top connection, spall; bottom of column, cracks.
	G10	Top connection, spall.
Penthouse	G3	Top connection, hairline crack.
Roof	G8	Top connection, beam, side spall.

GROUND MOTION AND RESPONSE

There were no accelerographs at or near the building, which is located between the instrumented locations at Pacoima Dam, the Holiday Inn on Orion Avenue, and Castaic Dam. A summary of approximate spectral response values for this building is given in table 2. These values are obtained from preliminary elastic single-degree-of-freedom relative velocity response spectra determined from the records at these instrumented sites and assuming 5-percent damping. Computations, using some simplifications, indicate that this structure would have a natural period of about 1.25 seconds in both east-west and north-south directions. Using this value of T, the spectra for Pacoima Dam indicate maximum accelerations of 0.65*g* in the S.74°W. direction and 1.04*g* in the S.16°E. direction (table 2). Similarly, the Holiday Inn response spectra give maximum accelerations of 0.45*g* in the north-south direction and 0.30*g* in the east-west direction.

Elastic analyses of simplified planar elements using time-history records and 5-percent damping were run by the University of Illinois. In these analyses, the shear walls were assumed to be simple cantilevers fixed against rotation at the base. In the east-west direction, a single wall was analyzed assuming one-fourth of the total building mass of each floor lumped at that level. In the north-south direction, the walls on lines 1 and 3 were linked at the floor-lines and the system assigned one-half the total floor building mass. Two analyses were made using the Pacoima Dam S.74°W. record. One used the record as

Table 2.—Approximate elastic spectral relative velocity

[5-percent damping]

Location	Direction of record	Period (T) = 1.25	
		Relative velocity (in./sec)	Fraction of gravity
Pacoima Dam	S.74°W.	50	0.65
Do	S.16°E.	80	1.04
Holiday Inn	North	35	.45
Do	West	23	.30
Castaic	N.21°E.	10	.13
Do	N.69°W.	16	.21

NOTE.—Relative velocity is the relative velocity for a period of 1.25 seconds as measured from spectral relative velocity curves found elsewhere in this volume.

Fraction of g given is $\frac{0.0162}{T}$ times the relative velocity.

recorded, and the other used the measured accelerations scaled down to a maximum of 0.4g. A third analysis in each direction of the building used the north-south record of the Holiday Inn at Orion. These records were used as being those most nearly bracketing the behavior at the site of the building. Thus, the larger S.16°E. Pacoima Dam record was not used. From the analyses, the base shear in the north-south direction is 0.63g based on the full Pacoima record and is 0.23g based on the north-south Holiday Inn record (table 3). In the east-west direction these values are 0.76g and 0.23g, respectively (table 4). For all the earthquake records used, the maximum acceleration occurred when the building was vibrating primarily in the first mode of vibration.

The structure also has been analyzed for the seismic forces required by the 1971 Los Angeles City Building Code for a K = 1.0-type building. The base shear was 0.045W in each direction, where W is the dead load of the building on each floor. The base shear was distributed to each level in conformance with code requirements and is used as the basis of the "ratio to code" values included in tables 3 and 4.

The ultimate capacities of several of the critical shear walls in compression, tension, and shear have been computed and the margins of safety obtained for each. The margin of safety in this case is the ratio of the ultimate capacity of the wall to the stress resulting from code forces. It is of interest to note that the margin of safety in tension is slightly below

Table 3.—Elastic analysis of simplified planar elements using time-history records and 5-percent damping—N-S walls [1]

Floor level	ΣWeight [5]	Pacoima S.74°W. full [2]				Pacoima S.74°W. (0.4g max.) [3]				Holiday Inn N-S [4]			
		Maximum shear [6]	Time [7]	Percent g [8]	Ratio to code [9]	Maximum shear	Time	Percent g	Ratio to code	Maximum shear	Time	Percent g	Ratio to code
7	434	1,087.52	8.60	2.50	19.2	347.87	8.60	0.81	6.16	300.01	11.92	0.69	5.31
6	1,416	1,689.31	3.70	1.20	17.0	540.37	3.70	.38	5.43	699.48	11.92	.50	7.04
5	2,398	2,154.91	4.18	0.89	15.2	689.30	4.18	.29	4.84	920.70	11.88	.38	6.48
4	3,238	2,524.69	4.20	.78	13.9	807.59	4.20	.25	4.45	1,092.76	11.84	.34	6.01
3	4,078	3,123.25	3.62	.77	14.9	999.05	3.62	.24	4.79	1,250.04	11.84	.31	5.98
2	4,918	3,479.19	3.62	.71	15.2	1,112.91	3.62	.23	4.88	1,328.23	11.84	.27	5.82
1	5,926	3,454.49	3.62	.63	15.6	1,200.97	3.62	.20	4.98	1,408.59	11.80	.23	5.85

[1] Analyses by University of Illinois.
[2] Pacoima Dam time-history record as recorded.
[3] Pacoima Dam time-history record scaled to 0.4g maximum acceleration.
[4] Holiday Inn at Orion Avenue time-history record as recorded.
[5] Summation of weight in kips associated with two wall-linked system of all levels at or above floor under consideration. Half of total building weights.
[6] Maximum shear from analysis in kips.
[7] Time from start of record in seconds.
[8] Percentage ratio of maximum shear to weight at and above level under consideration.
[9] Ratio of maximum shear from elastic analysis to shear required by 1971 UBC.

Table 4.—Elastic analysis of simplified planar elements using time-history records and 5-percent damping—E-W walls

Floor level	ΣWeight	Pacoima S.74°W. full				Pacoima S.74°W. (0.4g max.)				Holiday Inn N-S			
		Maximum shear	Time	Percent g	Ratio to code	Maximum shear	Time	Percent g	Ratio to code	Maximum shear	Time	Percent g	Ratio to code
6	707	1,067.28	6.22	1.50	21.0	341.40	6.22	0.49	6.72	279.34	14.88	0.39	5.47
5	1,127	1,215.36	3.48	1.08	16.6	388.76	3.48	.35	5.29	404.42	14.88	.36	5.52
4	1,547	1,371.73	3.48	0.89	14.9	438.78	3.48	.29	4.78	487.46	14.88	.31	5.32
3	1,967	1,536.19	4.02	.78	14.4	491.39	4.02	.25	4.60	548.79	14.84	.28	5.15
2	2,387	1,830.74	6.30	.77	15.6	585.61	6.30	.24	4.99	618.68	14.84	.26	5.27
1	2,858	2,149.24	3.56	.76	17.3	687.49	3.56	.24	5.51	665.29	14.84	.23	5.32

NOTE.—See table 3, except ΣWeights are ¼ total building weights associated with a single cantilevered wall.

2.0 in two of the walls, while the lowest margin of safety in shear and compression was 2.5 (table 5). Thus, if the distribution of the base shear throughout the building were to follow the distribution set up by code criteria, it would require a base shear of 2 to 2½ times the code shear to reach capacity. With the same masses, these factors can be used as multipliers of the C factor and related to ground acceleration. Thus, the base acceleration must have been 0.09*g* or better to reach the ultimate capacity of the walls.

Table 5.—Seismic shear walls

[Margins of safety to code seismic]

Wall	Floor	Compression	Tension	Shear
Lines 1 and 10	5	5.95	2.19	4.0
	2	....	2.2	2.5
	1	2.5	1.99	2.5
Lines 3 and 8	5	5.2	1.80	4.3
	2	....	1.7	2.7
	1	3.3	2.10	2.9
Lines A and G	5	9.3	3.8	4.9
	2	....	3.0	3.0
	1	3.3	2.3	3.1

NOTE.—The margins of safety represent the ratio of the ultimate stress capacity of the wall at the location indicated to the stress resulting from the forces specified by the 1971 Los Angeles Building Code.

Based on a visual estimate of the extent of damage due to motions in each direction, it is estimated that deformations actually were on the order of three to four times those reached at ultimate capacity stress. Assuming the earthquake response for the actual building would produce deformations approximately of the same magnitude as though it had remained elastic, the base acceleration of the elastic system would be from 0.27*g* to 0.36*g* (0.09*g* times 3 or 4). This is reasonable because the elastic analysis by the University of Illinois indicates a maximum acceleration between 0.23*g* and 0.76*g*. The elastic spectral values of between 0.45*g* and 1.04*g* for the single-mass system thus is approximately 1.4 to 2.0 times the response of the distributed system assumed in the time-history analyses.

Considering all the factors available, it is estimated that the maximum ground acceleration was in the order of 0.4*g* to 0.5*g*; the maximum acceleration of a fully elastic model of the building as a whole, with 5-percent damping, would be approximately 0.35*g*; and, of course, the actual acceleration on the building as limited by member capacities was approximately 0.10*g*.

CONCLUSIONS AND RECOMMENDATIONS

A review of the details and the damage observed suggests the following conclusions and recommendations:

1 No evidence was found to contradict a statement that, in general, the building was designed and constructed to conform to the governing building code.

2 Horizontal movement and some crushing occurred at horizontal construction joints. This is complicated in this case by the intrusion of lower strength lightweight concrete into higher strength stone concrete.

3 Spalling and crushing occurred in lap-splice areas of vertical rebars in columns. Special ties at these locations should be considered. The whole question of confinement of concrete by ties at compression and tension splices needs further study. Similarly, damage to boundary elements of shear walls in tension casts some question on the lap-splice criteria used, particularly where seismic forces are to be resisted. Does confinement offer the practical solution to this? Perhaps boundary elements of shear walls, at least at splices, should have confining reinforcement similar to that now recommended for ductile columns, or possibly always should be designed as tension splices.

4 In this building, the east-west shear walls were interconnected by slab thickness ties only, which might well simulate hinged conditions. In the north-south direction, shear walls were interconnected or coupled by girders, plus a fairly stiff diaphragm slab, and these girders obviously attempted to assume or resist the angular rotation of the shear walls at their intersection. In general, design engineers should give particular attention to this in design. This problem is complicated by slab diaphragm deflection and inelastic wall deformations. The slab shear deflections should be considered.

5 The deflections resulting from the shear walls deforming inelastically can produce deformations in the vertical load frame that might reach ultimate stress conditions. It is possible that consideration should be given to requiring all columns to be designed for shear to support the maximum possible end moments and also to providing minimum confinement at ends of columns.

6 Considerable study recently has been given to shear stresses in shear walls of various aspect ratios subjected to reversing bending and direct stresses.

More research is desirable on this subject, and the behavior of the shear walls in this building will serve as a valuable subject of study. This is particularly true for a study of the inelastic behavior of shear walls. The effects on the resistance of shear walls of reinforcing in both horizontal and vertical directions should be evaluated.

7 Horizontal cracks were found at cold joints. Obviously, the bond here was not perfect. This is not a new problem. Keys can give only a 50-percent shear area at best. Joint cleaning is supposed to be the answer. Is it practical to get 100-percent bond at a cold joint? If not, what should be done and how does this affect the earthquake-resisting capacity of the wall? Further study is required for this problem.

8 In general, the building response was within that envisioned by design philosophy. The behavior was such that it should be used by engineers to assist in setting appropriate design levels for future buildings.

Holy Cross Hospital (23)

15031 Rinaldi Street, Los Angeles

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Photographs provided by Los Angeles City Department of Building and Safety unless otherwise noted.

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GENERAL DESCRIPTION

The buildings comprising the Holy Cross Hospital are located at the intersection of Rinaldi Street and Indian Hills Road in the city of Los Angeles. The Lower Van Norman Dam is approximately 1 mile to the west. The topography in general is gently sloping (3½ percent up toward the north and east). Ground cracking or fissuring was not observed at the site after the earthquake. An aerial view of all buildings in the immediate area is shown in figure 1.

The main building was constructed with a seven-story tower (fig. 2), a three-story wing to the north, and one-story wings to the east and west. A single-story basement was built under the main tower. The building permit was issued on May 7, 1959, and the certificate of occupancy was issued on March 21, 1963. The codes in effect at time of completion of design were the 1959 Los Angeles City Code and U.S. Public Health Service Regulations, Part 53.

About 550 feet to the northeast of the main hospital building is a one-story continuing care unit. The plans for this building are dated in 1961. This structure was constructed in the shape of a cross with four wings about 50 feet wide intersecting in the center. The length of the wings, tip to tip, is about 200 feet. The west wing was constructed with a sliding joint so that transverse (north-south) seismic loads of the wing would be resisted entirely by walls within the wing. The longitudinal (east-west) seismic loads of this wing would be resisted by the remainder of the building. There is no basement. The roof framing consists of concrete joists, except in the west wing, which consists of concrete beams and slabs. Concrete shear walls occur on both sides of the centrally located corridors and at the ends of the wings. The exterior concrete columns are rectangular, projecting beyond the glass line like vertical fins. No moment resistance is provided at the top of these columns. The building is supported on spread

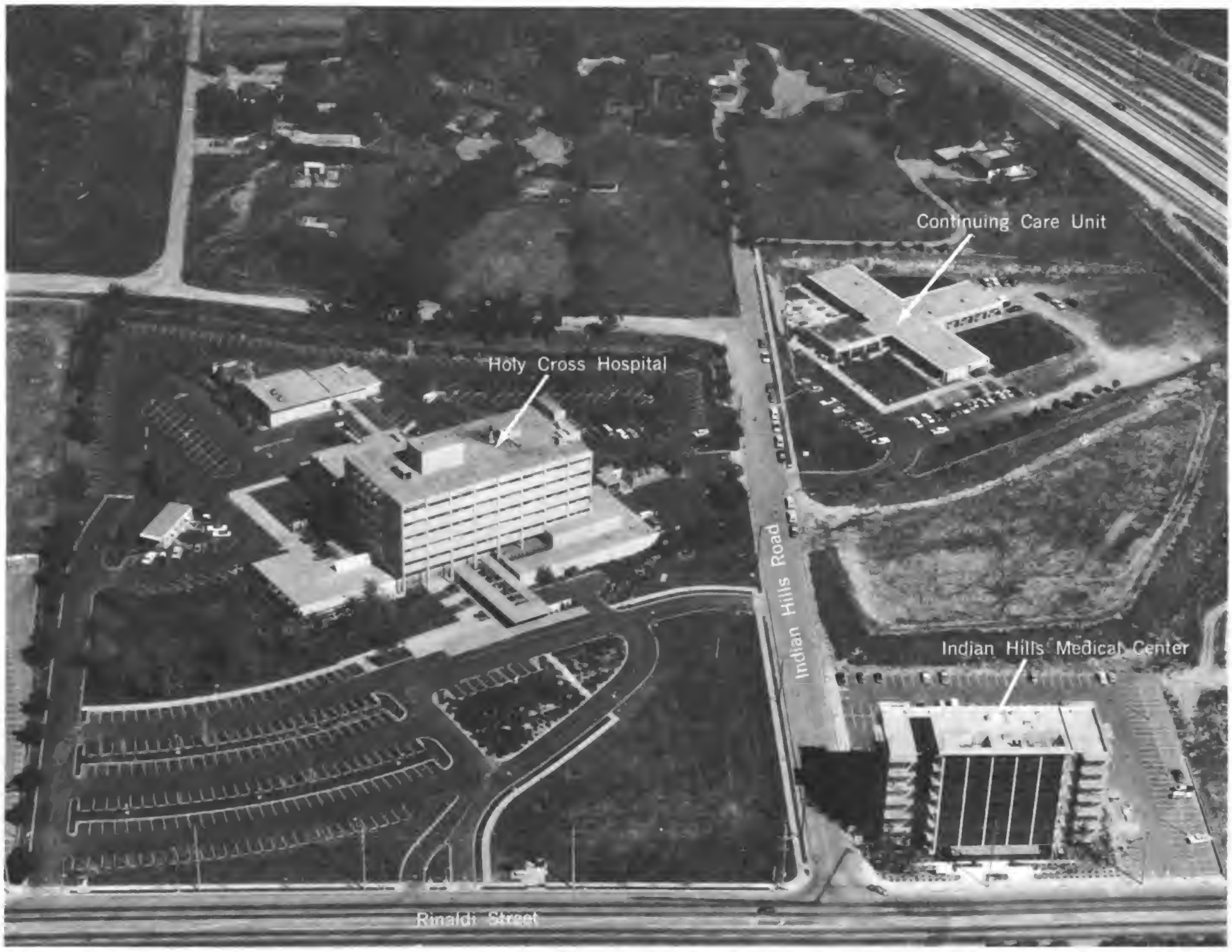


Figure 1.—Holy Cross Hospital. Aerial view of intersection of Rinaldi Street and Indian Hills Road, looking north. The buildings in the lower right are in the Indian Hills Medical Center. The other major buildings comprise the Holy Cross Hospital complex. Earthquake Engineering Research Laboratory, California Institute of Technology photograph.

footings. The floor is a concrete slab on grade. This is a low building with a height of about 12 feet.

North and a little to the west of the main hospital building is a one-story service building, built at the same time as the main building. This building is composed of a boiler room approximately 61 by 50 feet in size and some smaller service areas to the east of the boiler room. The total length of the building is 118 feet. The boiler room area is about 18 feet high; the other areas are lower.

The roof framing over the boiler room consists of reinforced concrete girders clear spanning the 50 feet and supporting concrete slabs. There is a clerestory window belt just under the roof. These windows are

Figure 2.—Holy Cross Hospital. Exterior view taken from south.

about 4½ feet high, leaving only four concrete columns on the north and south sides of the building to resist the east-west seismic forces. These columns are 10 by 12 inches, with four No. 9 bars and No. 2 ties at 10 inches on center. They are turned weak-way in the east-west direction. The exterior walls are reinforced grouted lightweight concrete unit masonry. The standard unit size is 8 by 8 by 16 inches. The floor is a concrete slab on grade.

Some 300 feet to the southeast of the main hospital building is the Indian Hills Medical Center. This is a seven-story reinforced concrete shear wall structure which sustained significant damage. It is discussed in Building Report 22.

From the soils engineer report, made prior to design of the main building, it is noted that there was no fill found on the site. The natural soil consisted of nonuniform alluvial deposits of sand, silt, and clay in various proportions. The upper soils were relatively low in density and shearing strength. The deeper soils were more dense and had higher shear resistance. It was indicated that the water table level varied from 10 to 25 feet below the then existing ground level, with the lower depth encountered at the time of the report (1958). Piles were recommended for the main building because of expected high settlements that would result from the use of spread footings. Sixteen-inch-diameter drilled and cast-in-place friction piles were used, with a basic design capacity of approximately 100 kips when the effective pile length was 26 feet. Variations were used for pile length and cluster efficiency. One-third increases in values were permitted for combinations involving wind or seismic stresses. Special longitudinal reinforcing was added to piles where design uplift was found.

The main framing of the seven-story tower (figs. 3 through 10) consisted of concrete joists having a stem 14 inches deep and supporting a 3-inch slab. The joists framed to beams of the same depth, or to walls along the interior column lines and to spandrels on the exterior column lines. On the north and south exterior walls of the main tower, spandrels were 8 inches thick and located on the inside edge of the long face of 16- by 32-inch columns. The one- and three-story wings (figs. 4, 5, 11, 12, and 13) were of similar joist and beam construction for floors and roofs. The plans called for a basement slab on grade, 4 inches thick, with mesh reinforcement placed over a 6-inch base of No. 2 rock.

The piles, pile caps, ties, slabs on grade, and boiler house columns were specified to have concrete with an ultimate compressive strength at 28 days of f'_c = 2,500 psi using natural aggregates. For main building columns and shear walls, concrete with f'_c = 5,000 psi and natural aggregates was specified. Horizontal framing, including girders, joists, slabs, stairs, and spandrels was to use concrete with f'_c = 3,000 psi and lightweight aggregates of the expanded shale type. Reinforcing steel was specified to conform to ASTM A–5 (f_s = 20,000 psi), intermediate grade. Structural steel used for canopies and penthouse framing was to conform to ASTM A–7 (f_y = 33,000 psi). Typical interior partitions were made of steel studs and plaster. Ceilings were primarily of suspended metal lath and plaster construction.

The lateral force system was designed using 8-inch concrete shear walls in each direction (figs. 3 through 16). As most of the shear walls were not continuous from top to bottom, reliance also was placed on the concrete floor joist and slab system acting as a diaphragm to transfer shears at points of discontinuity. Both architecturally and structurally, the general layout and design solution does not represent an unusual building, but due to the discontinuities, the lateral force-resisting system was complex.

It has been noted that 3,000 psi lightweight concrete was specified for the floor system. However, for columns and shear walls 5,000 psi hardrock concrete was specified. In the actual construction, the 3,000 psi lightweight concrete was poured through the 5,000 psi hardrock vertical elements. Thus, these vertical elements contained a layer of concrete having lower strength and a different type of aggregate for the depth of each floor and roof system. The lateral force design considered this by using an allowable stress in shear walls based on the lower concrete strength. This type of layering is used frequently in buildings throughout southern California. This building was designed to provide for three stories, in addition to those actually constructed.

EARTHQUAKE DAMAGE

The damage to the structural system of the main building was quite general but more pronounced on the first four stories. Some of the damage details are as follows:

1 The west shear walls (figs. 16 and 17) were discontinuous below the second floor. The diaphragms

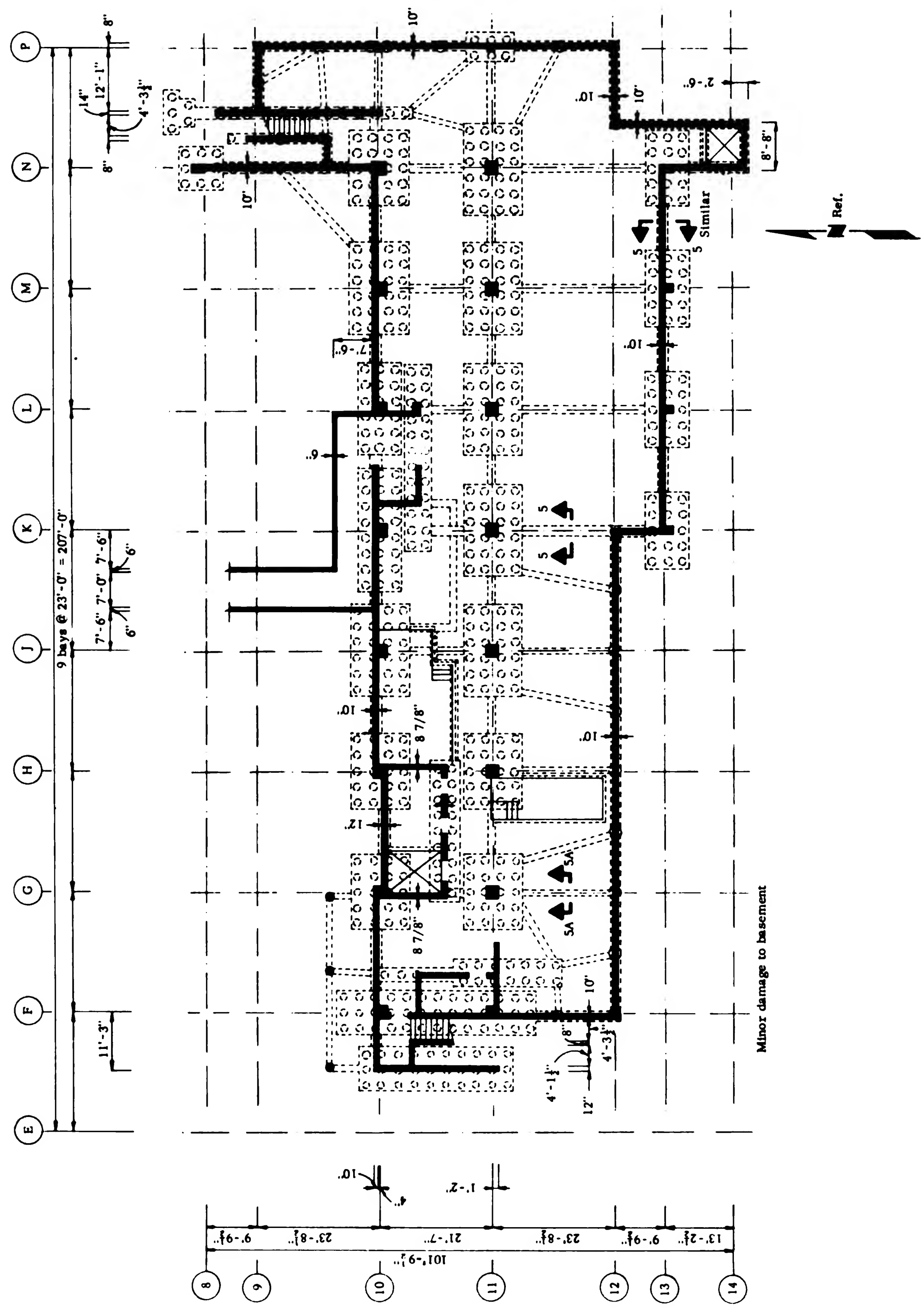
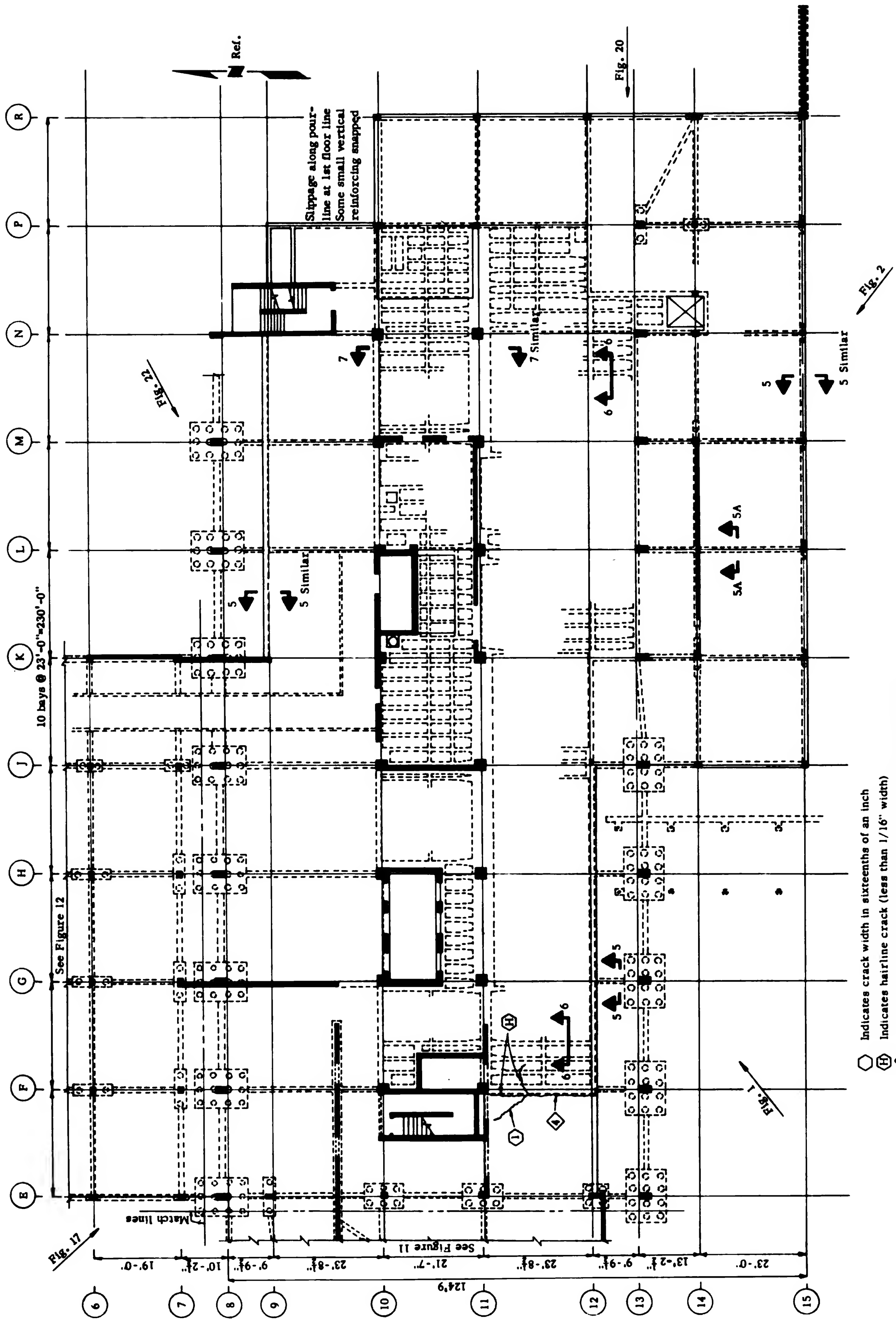


Figure 3.—Holy Cross Hospital. Basement floor and foundation plan.



Indicates crack width in sixteenths of an inch
Indicates hairline crack (less than 1/16" width)
Indicates vertical displacement at crack in sixteenths of an inch

Figure 4.—Holy Cross Hospital. First-floor plan (7-story tower and east wing).

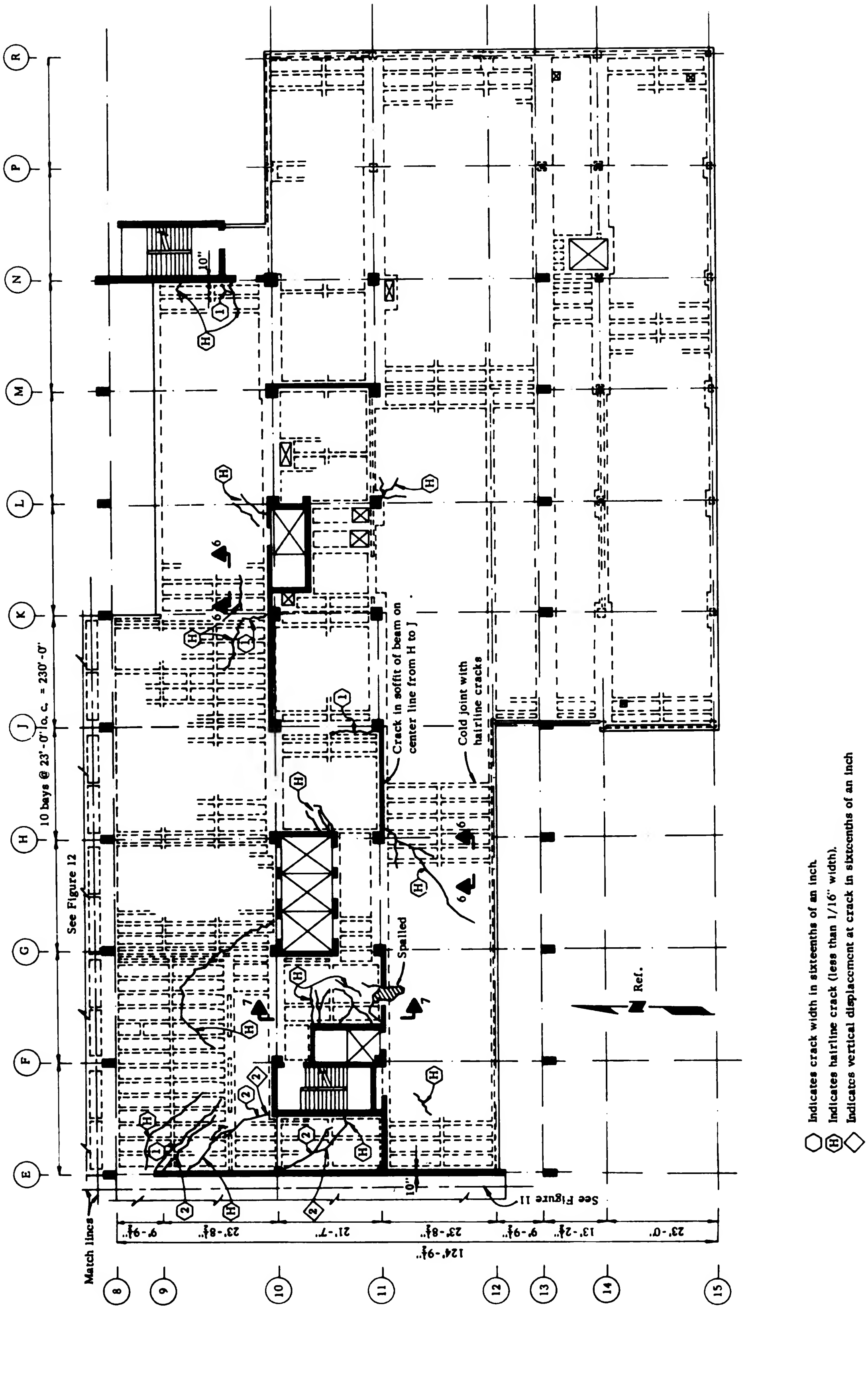


Figure 5.—Holy Cross Hospital. Second-floor framing plan (7-story tower and east wing).

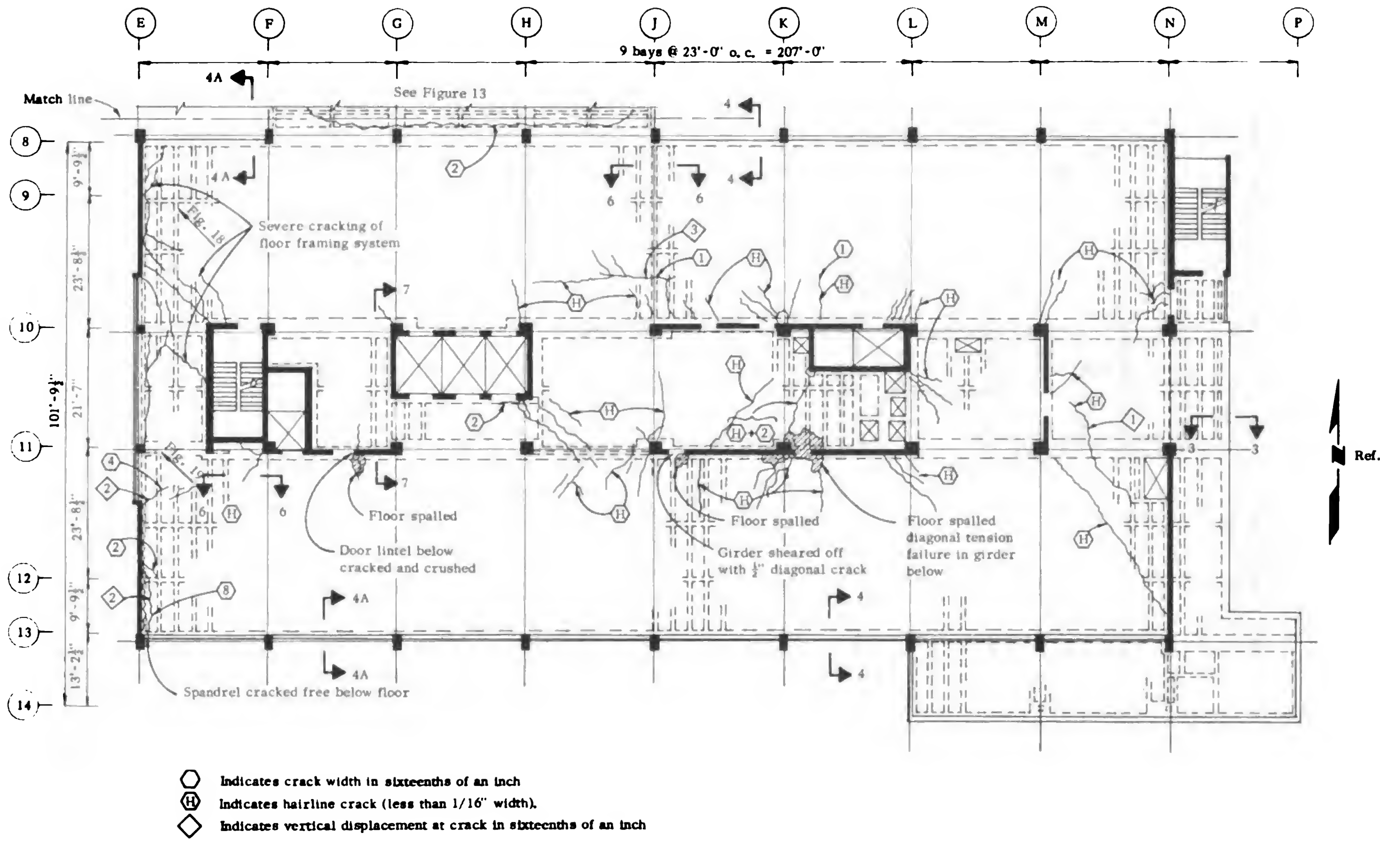


Figure 6.—Holy Cross Hospital. Third-floor framing plan (7-story tower).

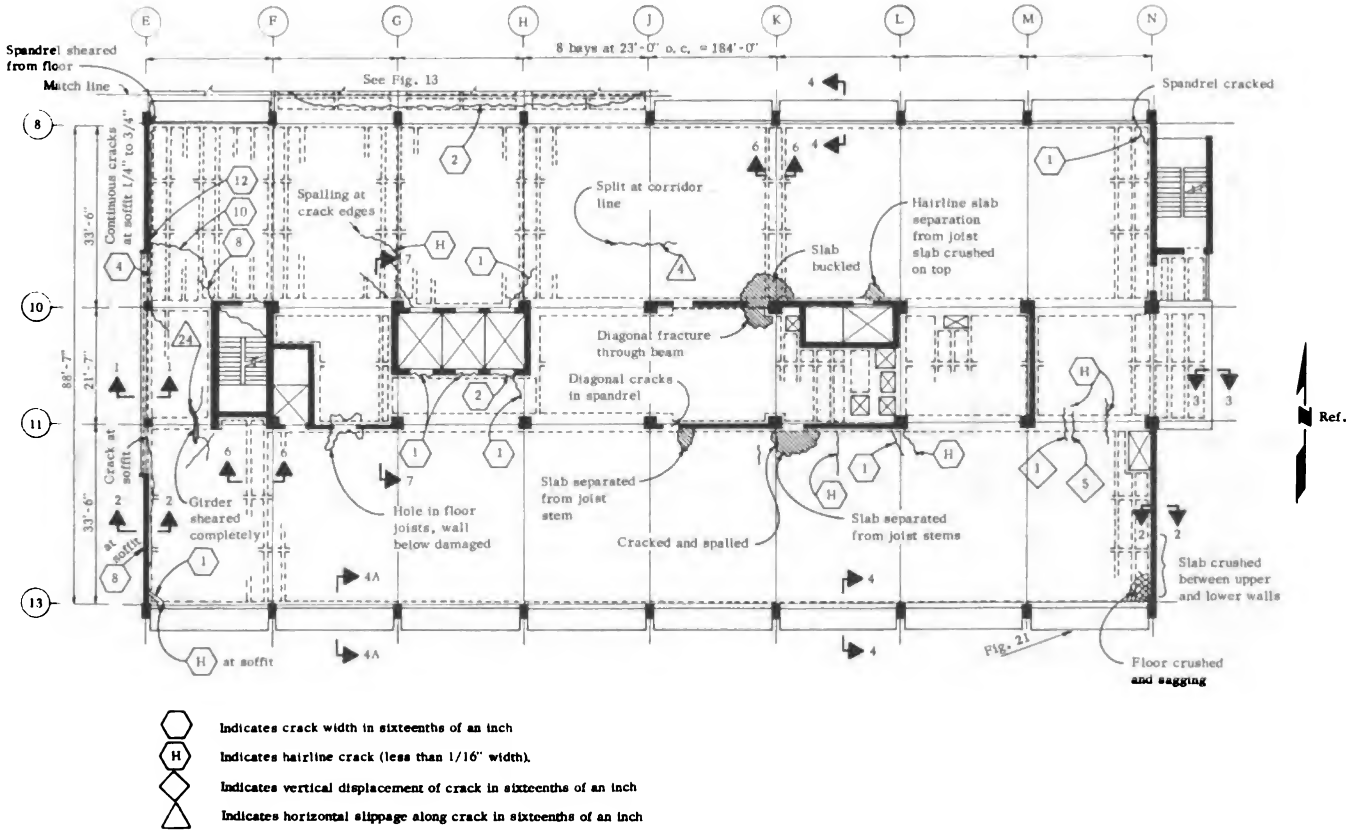


Figure 7.—Holy Cross Hospital. Fourth-floor framing plan (7-story tower).

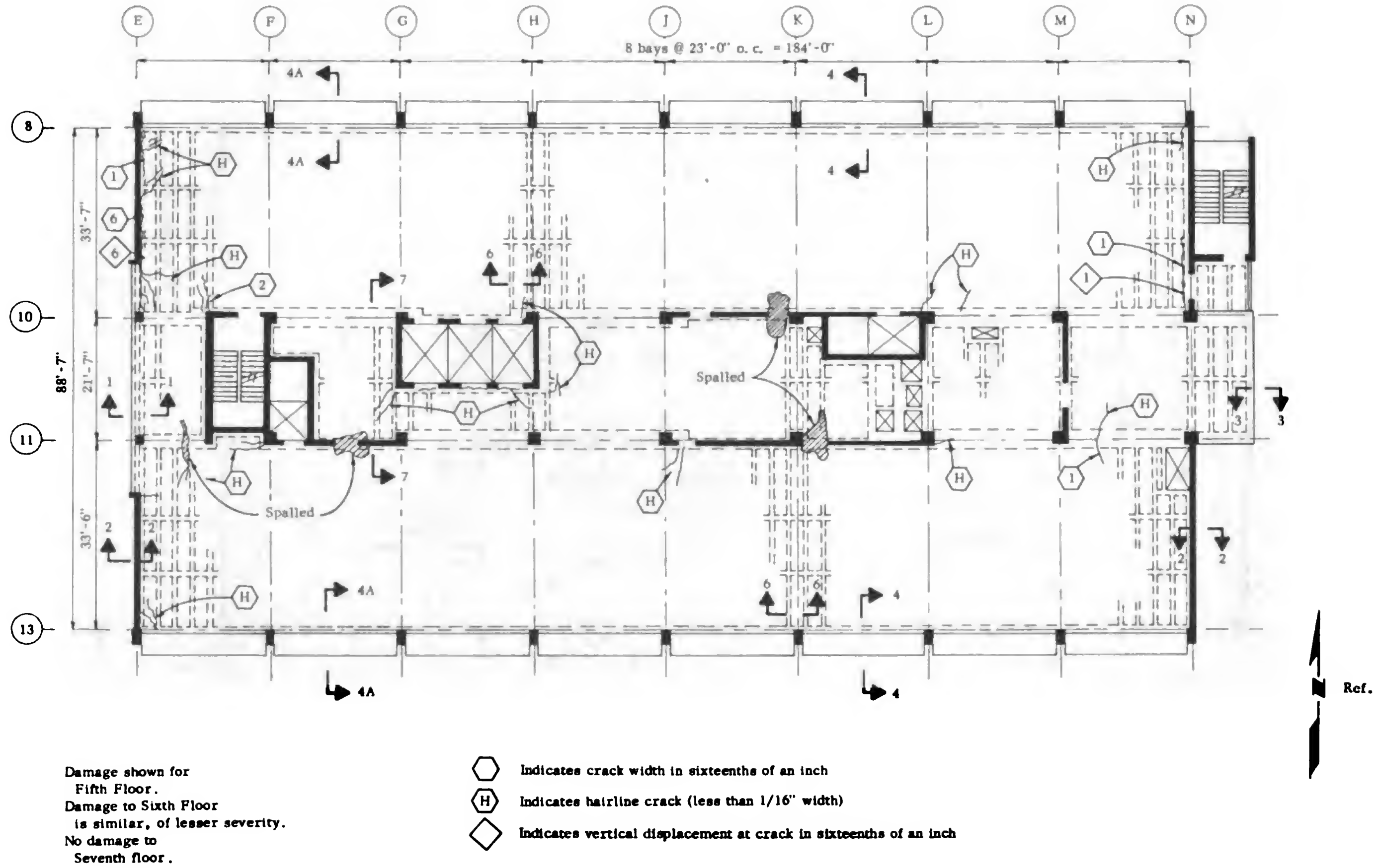


Figure 8.—Holy Cross Hospital. Fifth-floor framing plan (7-story tower). Sixth and seventh floors are similar.

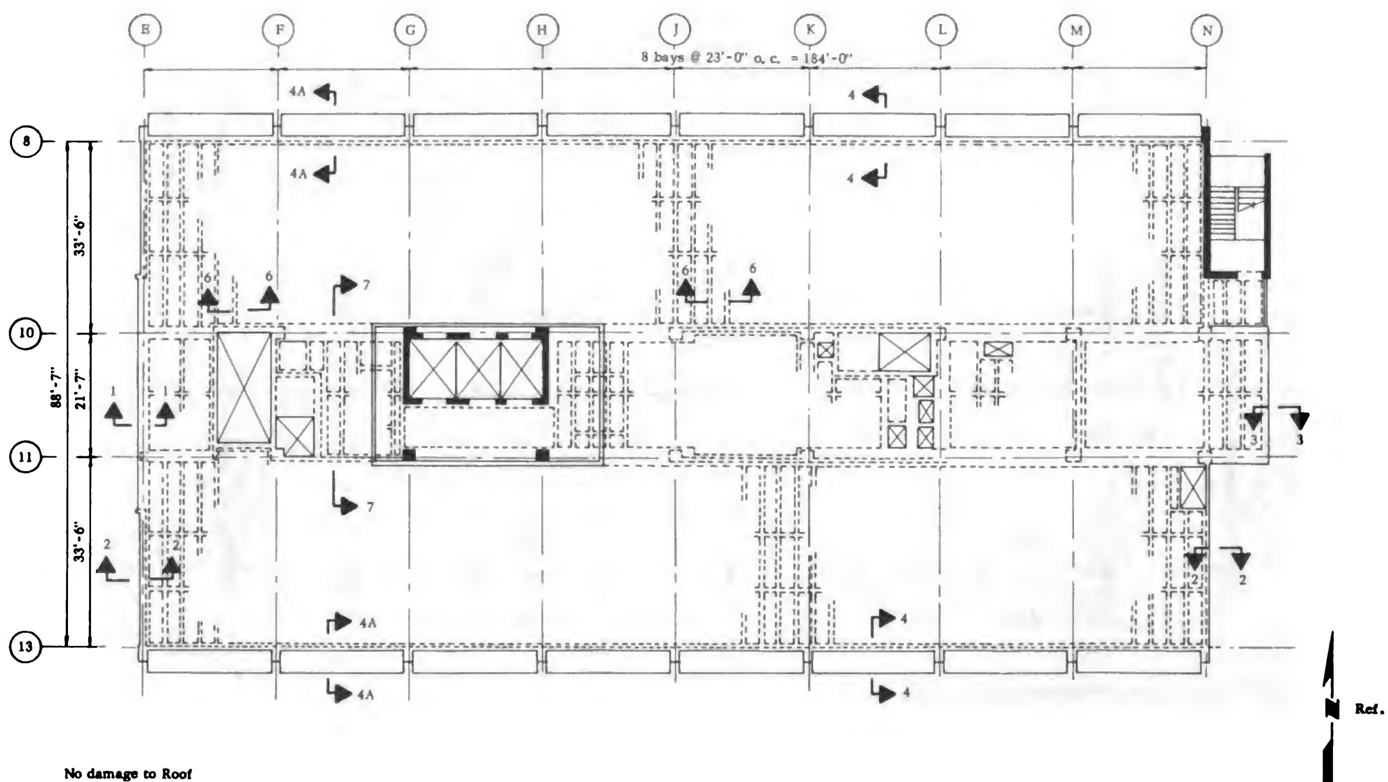


Figure 9.—Holy Cross Hospital. Roof framing plan (7-story tower).

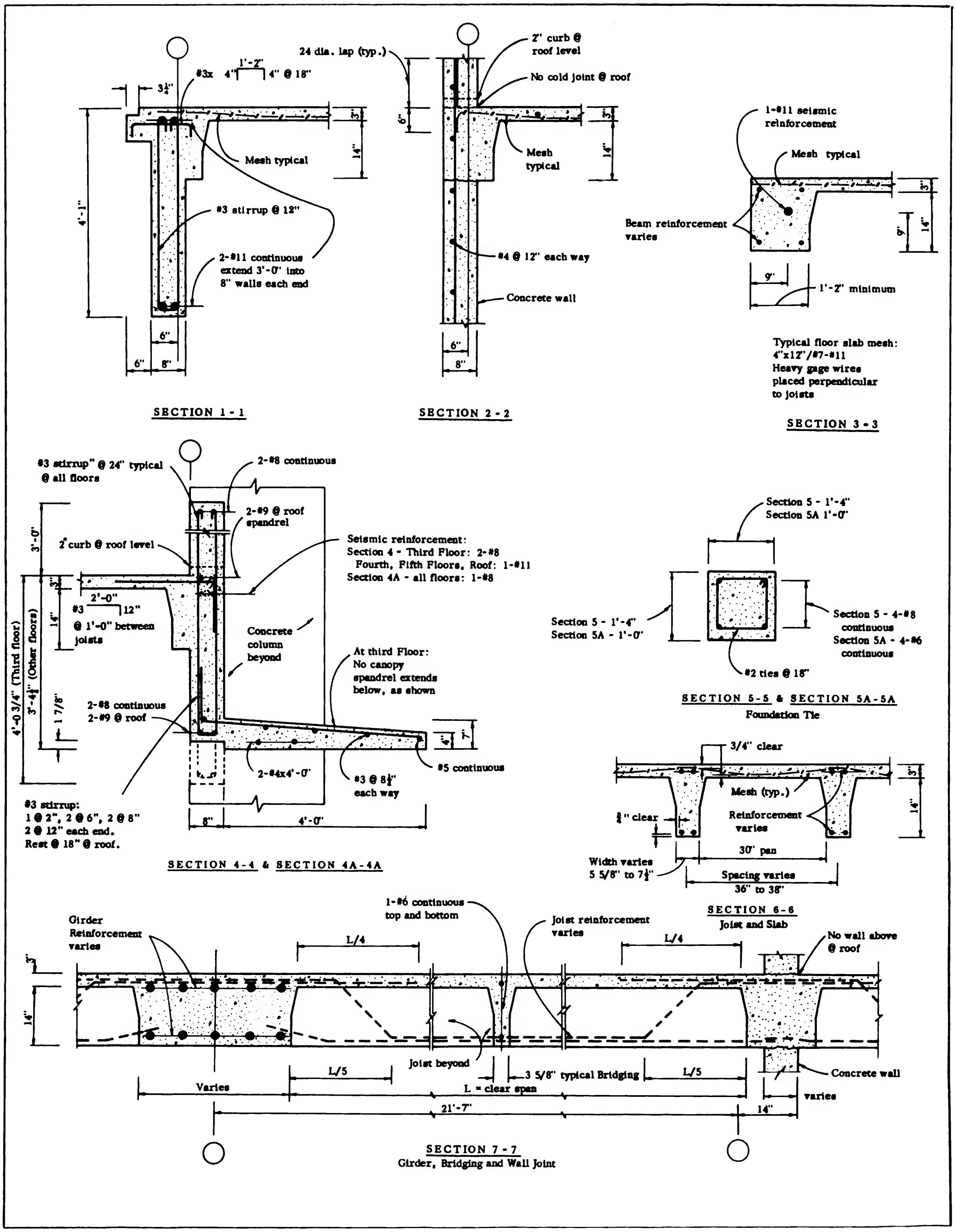


Figure 10.—Holy Cross Hospital. Sections.

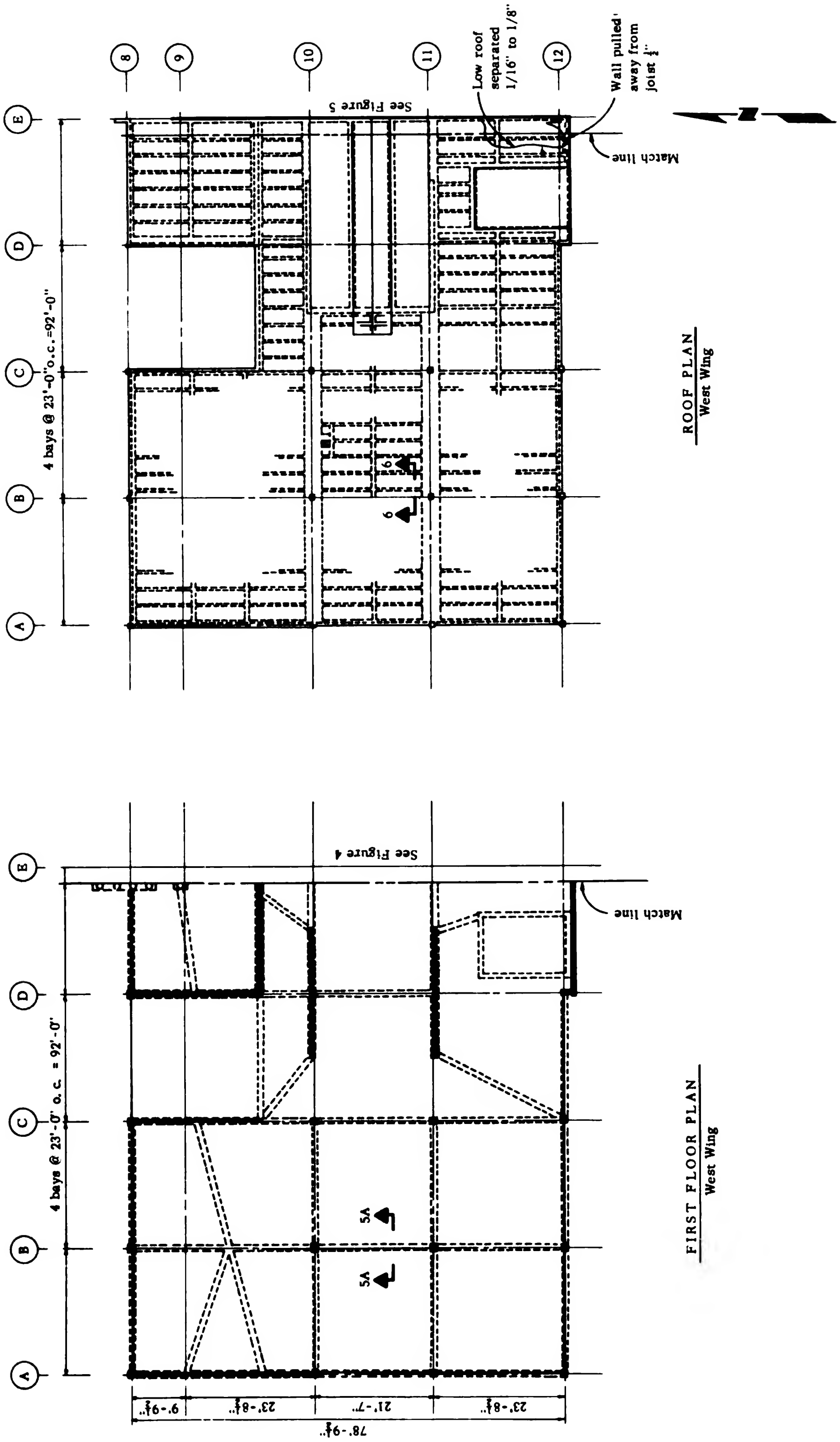


Figure 11.—Holy Cross Hospital. Framing plans, west wing.

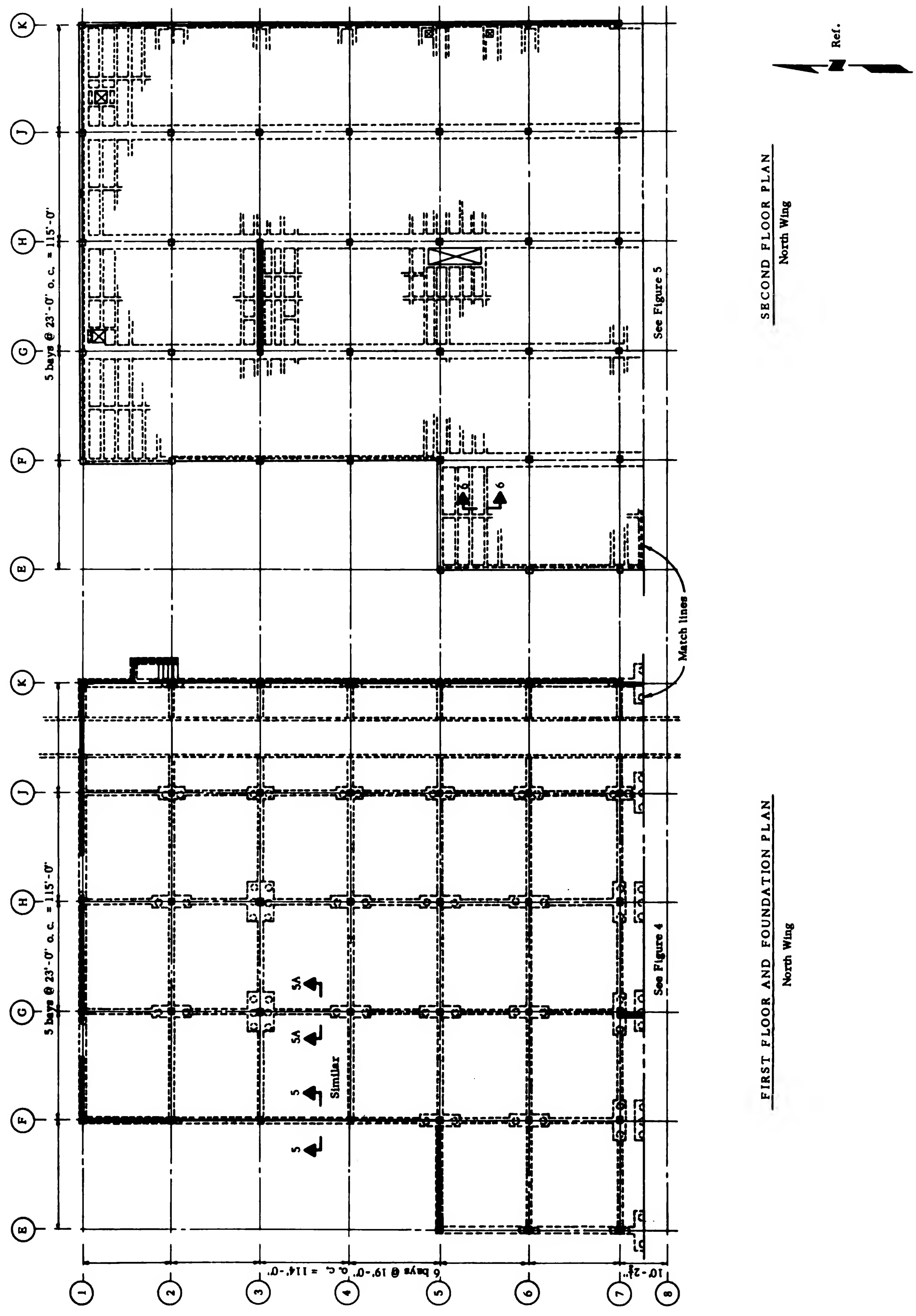


Figure 12.—Holy Cross Hospital. Floor framing plans, north wing. No damage to first and second floors.

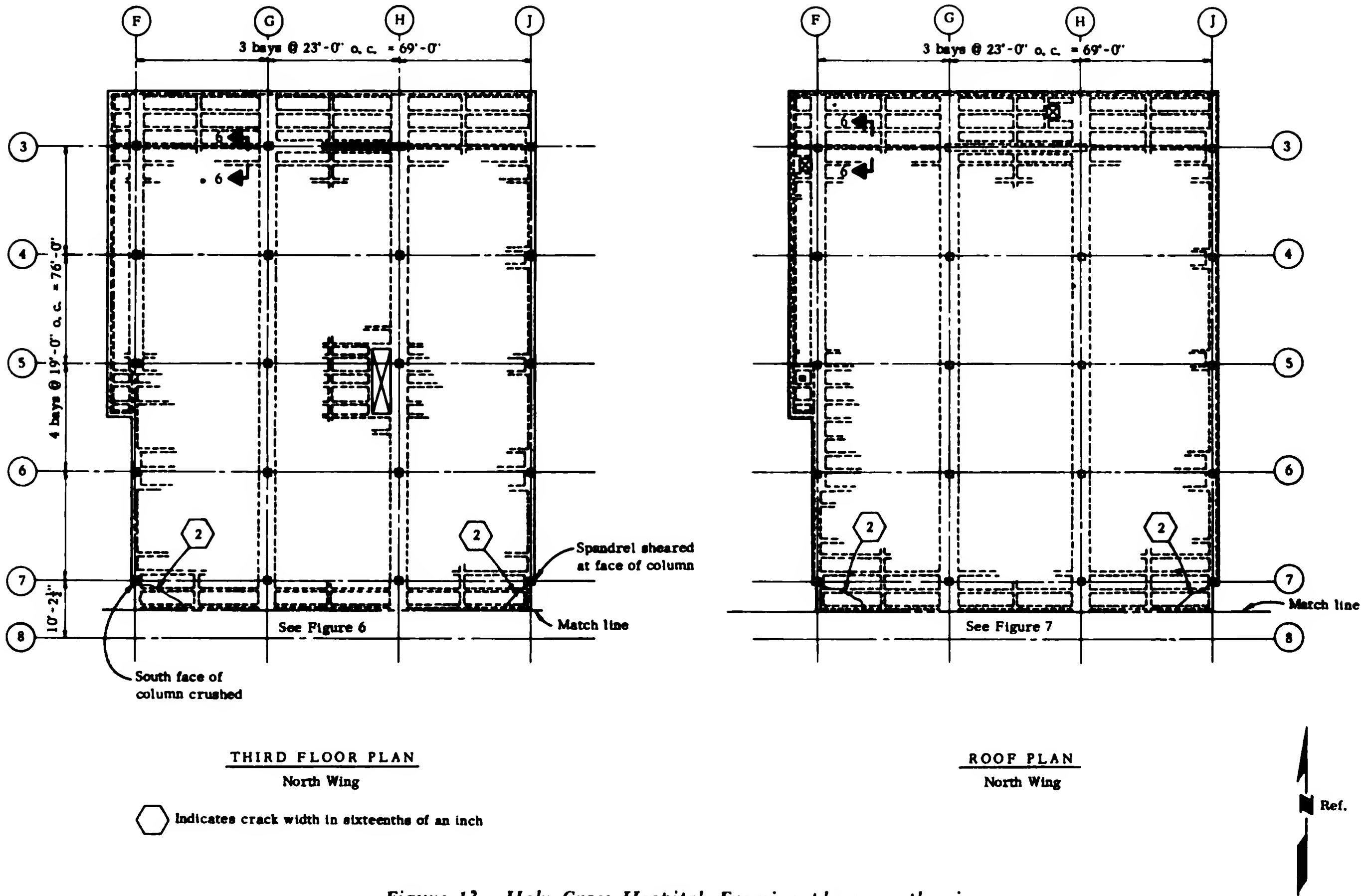


Figure 13.—Holy Cross Hospital. Framing plans, north wing.

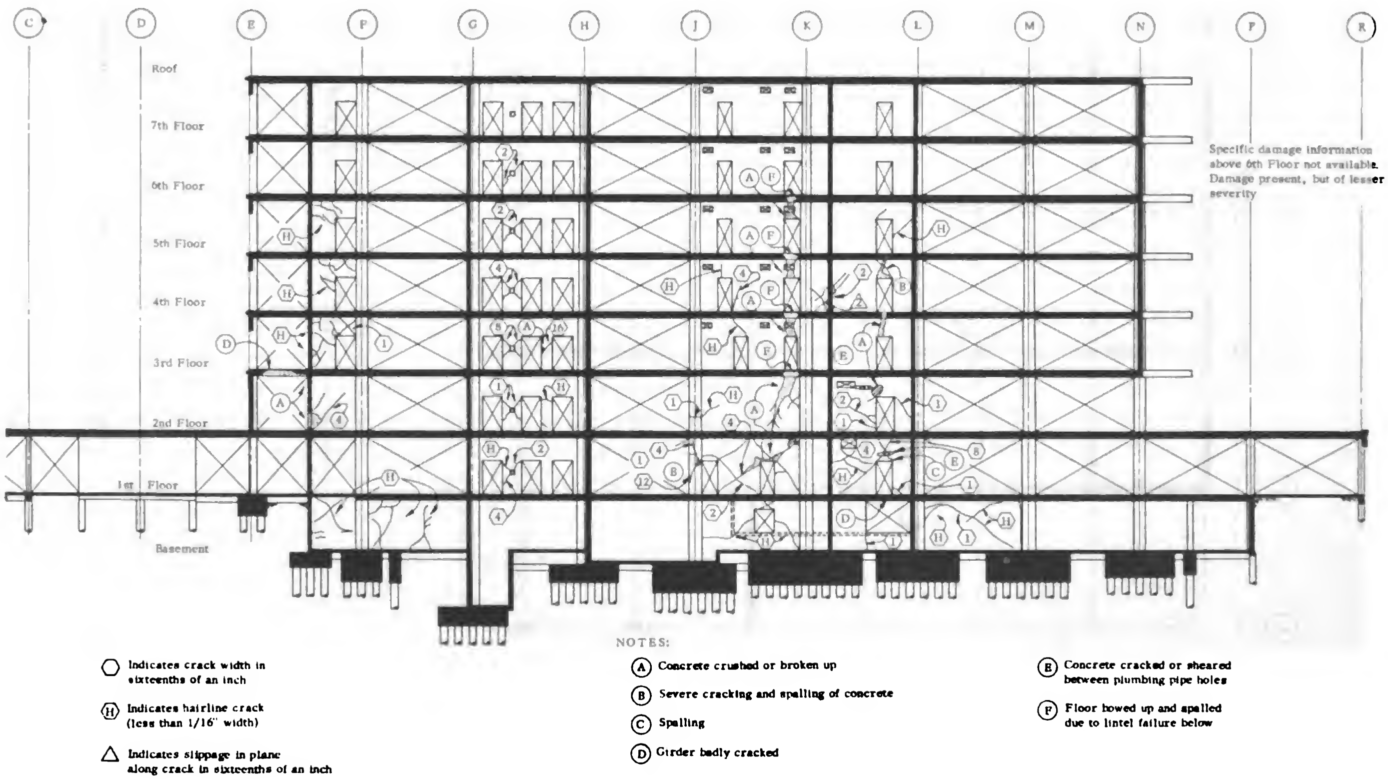
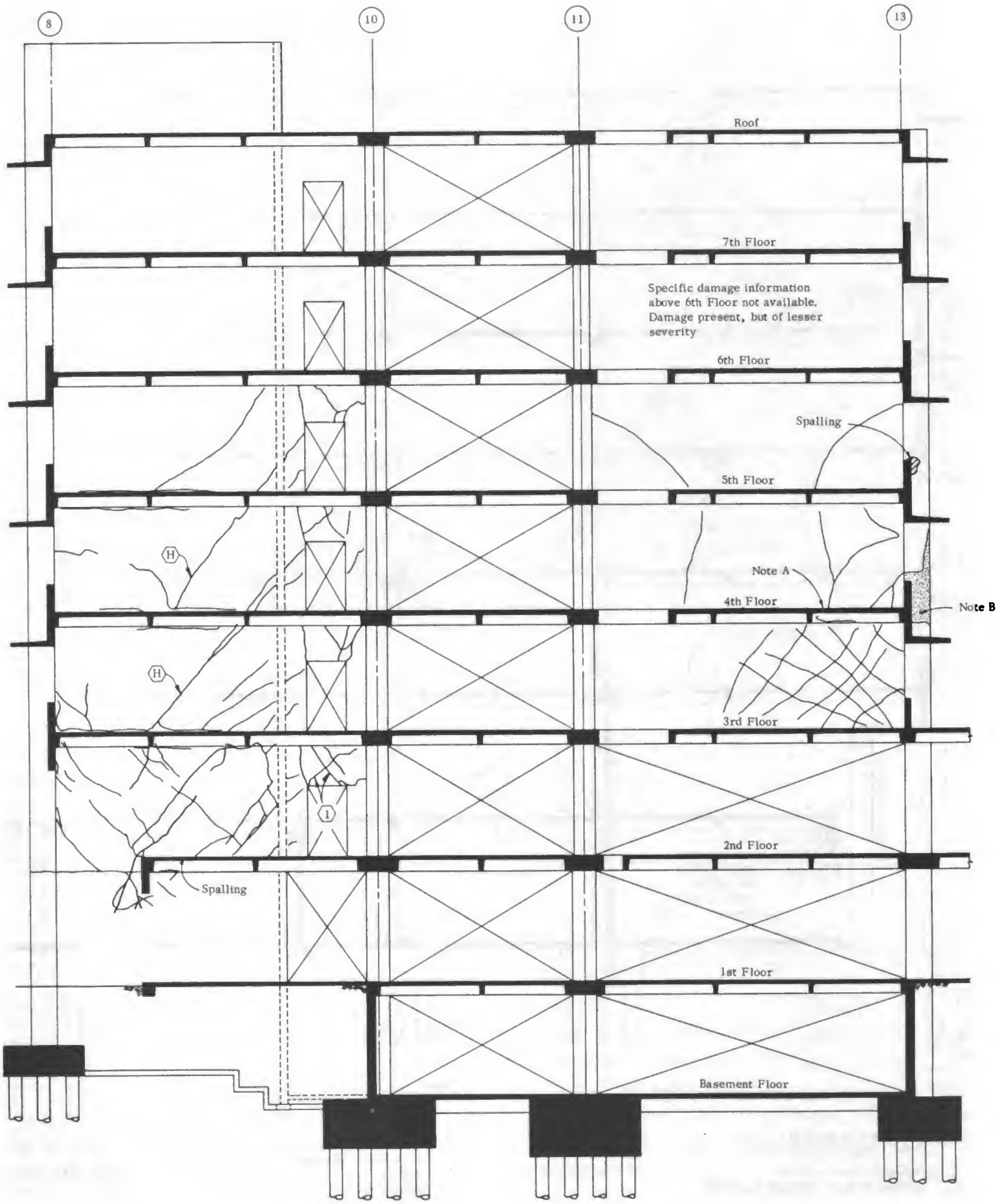


Figure 14.—Holy Cross Hospital. Section at grid line 10.

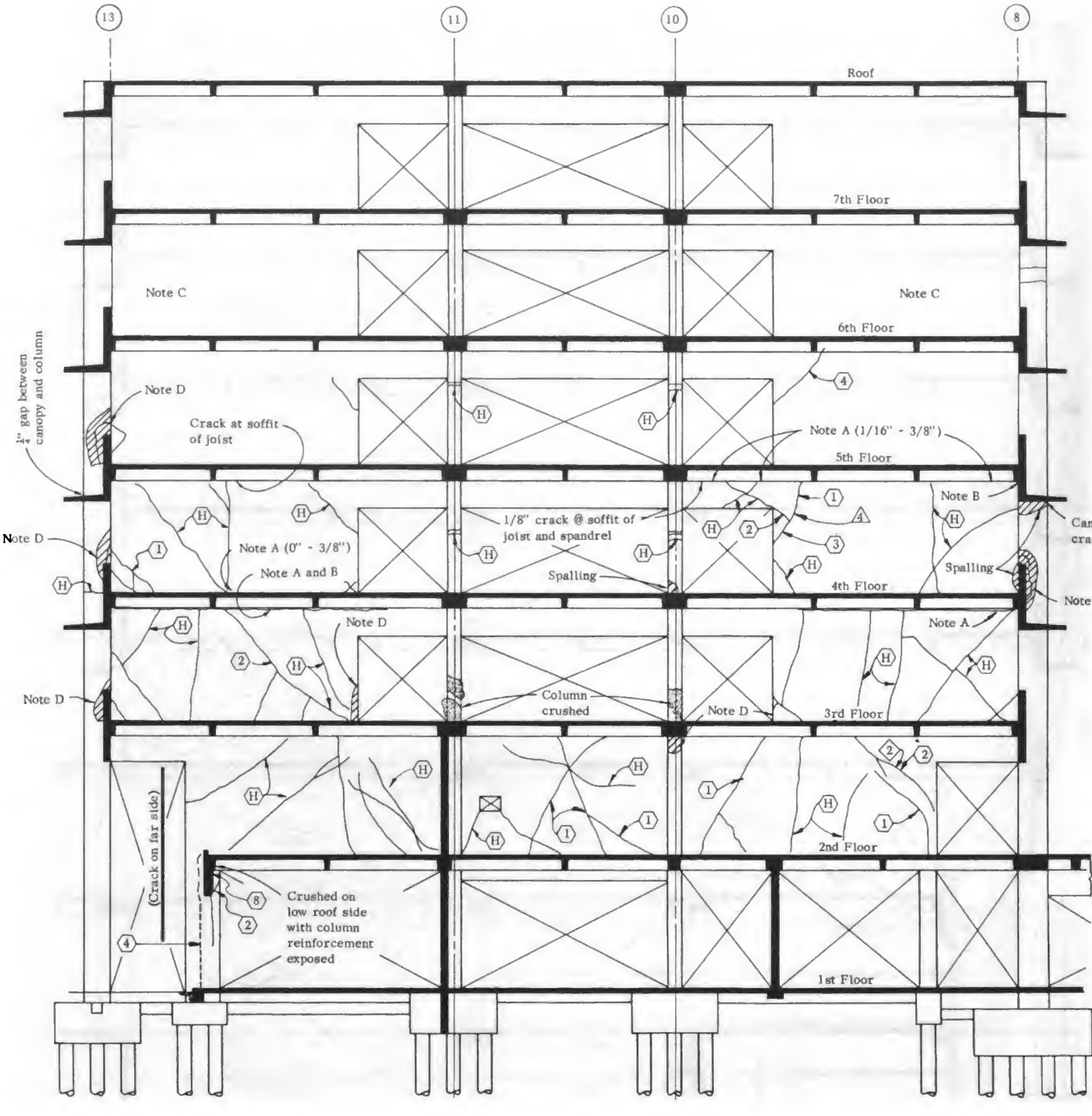


⬡ Indicates crack width in sixteenths of an inch

Ⓗ Indicates hairline crack (less than 1/16" width)

Notes:
A. Floor and wall separated 1¼"
B. Concrete completely crushed; reinforcement exposed

Figure 15.—Holy Cross Hospital. Section at grid line N (east wall, 7-story tower).



⬡ Indicates crack width in sixteenths of an inch

(H) Indicates hairline crack (less than 1/16" width)

◇ Indicates displacement of plane at crack in sixteenths of an inch

△ Indicates slippage in plane along crack in sixteenths of an inch

Notes:
A. Wall and floor or spandrel and floor separated by amounts noted, if given above
B. Crack at pour line
C. Specific damage information above 6th Floor not available. Damage present, but of lesser severity
D. Concrete spalled, reinforcement exposed

Figure 16.—Holy Cross Hospital. Section at grid line E (west wall, 7-story tower).

Figure 17.—Holy Cross Hospital, west wall.

Figure 18.—Holy Cross Hospital. Diaphragm failure at west wall.

were inadequate to redistribute horizontal loads on the second, third, and fourth floors, and were damaged severely in this area as indicated by cracks extending across the full width of the building (fig. 18).

2 Each of the west shear wall elements exhibited a cracking of the lightweight floor pour intrusion at the third-floor level, permitting large rotations to the walls above the break (figs. 16 and 17).

3 One of the west wall columns was shattered at the third floor (fig. 19).

4 The east shear wall crushed the lightweight floor pour intrusion at the south end of the wall at the fourth floor, and the wall shattered at the splice of the column reinforcement at the end of the wall that was acting as the shear wall vertical flexural reinforcement (figs. 20 and 21).

5 The east stair wall failed along the pour line at the first floor, snapping the smaller vertical reinforcing bars crossing the pour line (fig. 4).

6 The longitudinal (east-west) shear walls had numerous **X**-pattern failures over door openings, permitting the individual wall elements to displace or rotate with little restraint at the second-floor level and above. This failure pattern caused the door sills to hump up with respect to the jambs, so that doors were inoperative in many instances (fig. 14).

7 Excessive nonelastic movement of the shear walls produced deformations large enough to cause portions of the vertical load framing system to act as a moment frame so that columns were carrying seismic shears and moments. Due to deformations in the transverse (north-south) direction, the columns exhibited shear cracking. This cracking was most severe in the second, third, and fourth stories, decreasing above the fourth level (figs. 2 and 22). In the longitudinal (east-west) direction, many of the exterior spandrels were crushed in flexural compression and the column cover was shattered, exposing the reinforcing (fig. 16). This was most noticeable on the third floor.

8 The three-story north wing (a surgical and special treatment area) was, in general, constructed with only one shear wall. It was in the east-west direction in the north portion of the building. The rest of the lateral force was reacted by the shear walls in the main tower. Framing elements at the juncture of this wing and the main tower and the diaphragm slabs at the intersection were cracked

Figure 19.—Holy Cross Hospital. Column failure at E-11 (third floor).

Figure 21.—Holy Cross Hospital. Shear wall column, southeast corner.

Figure 20.—Holy Cross Hospital, east wall.

Figure 22.—Holy Cross Hospital. North side of tower.

(figs. 6, 7, and 13). This indicates that the north wing was in motion independently with respect to the main tower structure.

Even though the damage was rather extensive, the buildings satisfied the present basic intent and philosophy of seismic design, in that the building did not collapse under heavy ground shaking and was evacuated without casualties. The shattered columns mentioned in items 3 and 7 give an indication that the damage would have been greater had the high intensity of shaking occurred over a longer time interval.

Neither the continuing care unit nor the service building sustained significant structural damage during the earthquake. Some minor cracking was noticed in a few locations.

Examination of the building and design did not reveal any significant deviations from the requirements of the code in force at the time of construction.

GROUND MOTION, RESPONSE, AND DAMAGE EVALUATION

There were no accelerographs in or near these buildings. The location indicates that the ground motion would be intermediate to the instrumented motion at the Pacoima Dam and at the Holiday Inn on Orion Avenue. The maximum ground accelerations at these sites were 1.25*g* and 0.28*g*, respectively. Preliminary response spectra for these sites were used for the following discussions.

Calculations, using a simplified model of the complex lateral force-resisting system of the main tower as built, indicate that the fundamental period would be 0.65 second in the north-south direction and 0.80 second in the east-west direction. Small-amplitude transient vibration measurements were made just prior to demolition of the top two floors of the main seven-story tower. These measurements showed fundamental periods of approximately 0.66 and 0.80 second, respectively. Due to the damaged condition of the building at the time of these measurements, the validity of using them as being indicative of the period of the building prior to the earthquake is questionable. However, the original small-amplitude period only could have been somewhat shorter than these measurements. As the calculated periods of the original structure are nearly the same magnitude, it seems reasonable to use them as indicating the approximate fundamental period of vibration for strong-motion elastic response.

Table 1.—Approximate elastic spectral relative velocity

[5-percent damping]

Location	Direction	Period (T) = 0.65 second: Relative velocity (*in./sec*)	Period (T) = 0.65 second: Percent *g*	Period (T) = 0.80 second: Relative velocity (*in./sec*)	Period (T) = 0.80 second: Percent *g*
Pacoima Dam....	S.74°W.	50	1.25	38	0.77
Do..........	S.16°E.	35	0.87	42	.85
Holiday Inn......	North	32	.80	25	.50
Do..........	West	13	.33	12	.24
Castaic..........	N.21°E.	15	.37	13	.26
Do..........	N.69°W.	18	.45	23	.47

NOTE.—Percent *g* given is $\frac{0.0162}{T}$ times the relative velocity.

Using the calculated fundamental periods, the Pacoima Dam spectra with 5 percent of critical damping indicate a maximum ground acceleration of 1.25*g* in the S.74°W. direction and 0.87*g* in the S.16°E. direction. The Holiday Inn spectra give maximum accelerations of 0.80*g* in the north-south direction and 0.33*g* in the east-west direction. A summary of these values is given in table 1. These response values assume single-mass, single-degree-of-freedom systems.

The main tower building as built has been analyzed using the seismic forces required by the 1971 Los Angeles City Building Code for a K = 1.33-type building. From this, the base shears were 0.077W in the north-south direction and 0.078W in the east-west direction, where W is the dead load of the building above the first floor. The stresses at ultimate capacity in compression, shear, and tension of several of the more critical shear walls were compared to the stresses resulting from code forces. These comparisons are shown as margins of safety (i.e., capacity stress to stresses due to code forces) in table 2. In this table, it is noted that the first crack would be most likely in the spandrel located between lines J and L (fig. 14) on the first and second floors. As the other shear wall elements had greater capacity, they would have been capable of carrying additional shear after this first element had reached its capacity. In general, the margins of safety for most of the shear wall elements were in the order of 1.4 to 2.3. This would indicate that major distress would occur when stresses reached 1.8 to 2.0 times those resulting from code forces.

Based on a visual estimate of the extent of damage from motions in each direction, it is estimated that the deformation at the top of the building during the earthquake must have been three to five times

Table 2.—Résumé of smallest margins of safety [1] at selected elements using K = 1.33

Figure	Wall	Floor	Shear: Header	Shear: Pier
14	Between lines G and H	7th	8.1	13.5
		6th	3.5	4.0
		5th	2.3	2.9
		4th	1.8	2.5
		3d	2.1	2.9
		2d	2.3	2.6
		1st	2.0	2.5
14	Between lines J and L	7th	4.7	6.2
		6th	2.4	3.6
		5th	1.7	2.5
		4th	1.4	2.1
		3d	1.6	2.0
		2d	0.5	—
		1st	.5	—
16	Between lines 8 and 10, 11, and 13	7th	1.7	12.1
		6th	1.7	6.5
		5th	1.9	4.6
		4th	2.3	3.7
		3d	4.0	3.2
		2d	2.6	2.1
15	Between lines 11 and 13	7th	11.0	9.0
		6th	6.0	4.8
		5th	4.3	3.5
		4th	3.4	2.8
		3d	2.9	2.4

Figure	Column	Floor	Compression	Tension
16	On lines E and 10, E and 11	7th	17.6	3.6
		6th	8.3	2.3
		5th	5.5	1.9
		4th	4.1	1.9
		3d	3.5	1.8
15	On lines N and 13	7th	49.2	21.8
		6th	16.2	6.9
		5th	6.0	5.1
		4th	4.8	3.5
		3d	3.0	3.7

Figure	Diaphragm	Floor	Shear
16	On line E (N-S)	Roof	5.0
		7th	5.5
		6th	2.6
		5th	3.1
		4th	2.2
		3d	1.0

[1] Margin of safety is the ratio of stresses at ultimate capacity to stresses from forces required by code.

greater than that reached at ultimate stress in the wall piers. If it were assumed that a fully elastic response would produce the same deformations as were actually experienced, the base accelerations of the elastic system would be $0.078g \times 2 \times 3 = 0.47g$ to $0.078g \times 2 \times 5 = 0.78g$. The maximum elastic spectral values between $0.80g$ (Holiday Inn) and $1.25g$ (Pacoima Dam) for the single-mass system thus is about one or two times the above-mentioned response of an elastic system.

It is estimated from the information above that a best guess of the maximum ground acceleration at the site would be in the order of $0.4g$ to $0.5g$. The maximum acceleration of an elastic model of the building with 5-percent damping would be approximately $0.55g$. The acceleration at major cracking is indicated by stress analysis to be about $0.15g$.

A few specific items shown in table 2 are worthy of discussion. The low margins of safety indicated for figure 14 between lines J and L show up as large areas of damage (denoted by note A on the figure). However, this analysis did not indicate that the column and shear failures of the east and west walls are danger points.

On the west wall (fig. 16), the shear wall and column failures at the third floor do not indicate a particularly low margin of safety. The transfer of overturning stresses from the shear walls to the columns on lines 10 and 11 through the lintels could have produced unusual distribution of loads, including vertical gravity loads, once inelastic action took place. Much greater vertical loads could have been required to be carried by the columns than was indicated by the elastic analysis. This would have reduced materially the lateral load capability of the shear wall system, ultimately causing the column failures. The shear wall failures thus could have been a result of compressive flexure on the wall element once the columns had failed.

The east wall column and shear wall failure (figs. 15 and 21) is not pointed out by the analysis. The point of failure occurring at the point of splicing of the column bars on line 13 made this location an obvious weak point in the wall. The tower in the north-south direction did not have as many elements that would crack progressively as it did in the east-west direction. Also, the ground accelerations were obviously high enough to exceed the capacities of the walls.

The failure of the diaphragm adjacent to the west wall of the tower (see cracked area on fig. 6 near line E) is pointed out by the margin of safety of 1.0 indicated for the third floor. The failure of the diaphragm at this point could have been a contributing factor in the west shear wall failure, by forcing a redistribution of reactions to the wall.

Rehabilitation of the facility is proceeding on the north wing of the structure. The proposed modifications include separation of the north wing from the main portion of the tower, addition of concrete shear walls, and repair of structural elements by replacement or by epoxy repair of cracks. Also, a covered passage to the one-story continuing care unit is to be

provided. The main tower has had the top two stories removed, and it is anticipated that rehabilitation also will consist of the addition of some concrete shear walls, replacement of severely damaged structural elements, and repair of cracks by epoxy. The removal of the two top floors, and the fact that three stories were provided for but not built, will permit the addition of shear walls without requiring an increase in the foundation capacity.

The cost of the rehabilitation to the facility is estimated at \$4½ million, and represents a reduction of 20 percent of the floor area. The replacement cost for the main building as it was originally constructed is estimated to be about \$9½ million. Thus, the amount to be spent on rehabilitation is 48 percent of the worth of the building prior to the earthquake.

The continuing care unit structure had many shear walls in both directions, which resulted in a building with a period of about 0.1 second or less. The average shear stresses resulting from the code forces using 0.133W is approximately 20 psi in the east-west direction and 10 psi in the north-south direction. Assuming the ultimate capacity of the walls to be about eight times or more of the 20 psi, it is seen readily that quite high shears could be resisted.

Similarly, the service building has many shear walls with stresses resulting from very low code seismic loads. However, one interesting feature of the service building is the roof of the boiler area. As previously mentioned, there is a clerestory created by the high windows on the north and south sides of the building. This means that the columns in the clerestory provided the lateral force-resisting system for loads in the east-west direction. It was noted in the 1970 Peru earthquake that this type of construction was damaged very badly. For example, the columns in the Mercado Modelo in Huarmey were sheared and offset. The period of the boiler house roof system in the east-west direction is probably slightly less than 0.2 second. From the elastic spectra for the records at the Holiday Inn on Orion Avenue, the acceleration at that site for 0.1- or 0.2-second period would be approximately 0.5*g* in the north-south direction and 0.3*g* in the east-west direction, assuming a damping of between 5 percent and 10 percent of critical.

Assuming the columns pinned at the top, calculations show that the ultimate stress of the column would be 1.6 times the stress produced by code loads. Assuming a partial fixity at the top, this ratio could be increased 50 percent to 2.4, as the shear on the section is not critical. This then would require force of $2.4 \times 0.133 = 0.32W$ to reach ultimate. Some shears also would be carried by weak-way bending on the end walls. The clerestory also could have the effect of producing a two-level response system, which, for the short duration of this earthquake, may have reduced the maximum response.

CONCLUSIONS AND RECOMMENDATIONS

The survey of the damage to this building, considered with the benefit of hindsight, indicates that the following items could be altered to provide a better performance during earthquakes:

1 The use of $f'_c = 3{,}000$ psi lightweight concrete on floor pours in comparison to $f'_c = 5{,}000$ psi heavyweight concrete in the columns and shear walls undoubtedly influenced the transverse shear wall failures. ACI 318–71 (section 10.13) does not permit this disparity of strength in columns without special precautions being taken.

2 The failure in the east wall (figs. 20 and 21) indicates that a stagger or confinement of the lap splice of shear wall flexural reinforcement would assist in preserving this joint.

3 Increased use of shear reinforcing in wall elements over openings would have improved the behavior of the longitudinal shear walls. Holes for mechanical ducts through these lintels did not improve this situation. These shallow lintel elements are coupling vertical shear walls and should receive greater design recognition.

4 The north wing was of sufficient size to be a significant dynamic irregularity. Separation of this wing with an independent lateral force-resisting system would have improved the overall performance of the building.

5 Increased shear reinforcing in diaphragms, particularly at the bottom of discontinued shear walls, would have improved the performance of the floor systems in critical areas.

6 The deformations resulting from inelastic shear wall behavior also imposed high shears and moments on columns that were not designed to resist them. Consideration should be given to a requirement that vertical load columns be designed to resist shears produced by ultimate moments at the ends of the columns and that they be reinforced for confinement near the ends.

Olive View Hospital (24, 25, 26)

14445 Olive View Drive, Sylmar

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GENERAL OBSERVATIONS

The Olive View Hospital lies at the base of the San Gabriel Mountains within 0.3 mile of the Olive View fault and 0.7 mile south of the Hospital fault. The epicenter of the February 9, 1971, earthquake was located 6 miles northeast of the hospital. Although there was no apparent tectonic surface rupture of the soil within the boundaries of the property, there was considerable ground movement, both vertically and horizontally, and surface ruptures of the fault system were observed 3 miles southeast of the site.

Accelerograph records were obtained from instruments located at the Pacoima Dam, 3 miles to the east, and at the Holiday Inn, 7 miles to the south. The record from the damsite measured a maximum horizontal ground acceleration of 125 percent *g* and a vertical acceleration of 70 percent *g;* the other record indicated maximum ground values of 28 percent *g* and 17 percent *g*, respectively.

The new structures discussed in this report are located on the portion of the site that is a broad alluvial fan, formed at the mouth of Wilson Canyon. The soil materials consist of unconsolidated sands and gravels interspersed with rocks and large boulders to a depth estimated at 200 to 300 feet. The underlying strata are granitic bedrock.

For additional soils information, see the paper "Olive View Hospital Site Studies" in this volume.

The hospital complex (figs. 1 and 2) consists of many buildings. The older structures were constructed primarily of wood frame and unreinforced brick and hollow tile masonry. Because they were built prior to 1934, they were not designed to resist seismic forces, and many either suffered considerable damage and subsequently were demolished, or collapsed (fig. 3) and were removed later. For example, the original powerplant, built with a reinforced concrete frame and unreinforced brick masonry filler walls, was damaged beyond repair (fig. 4) and has

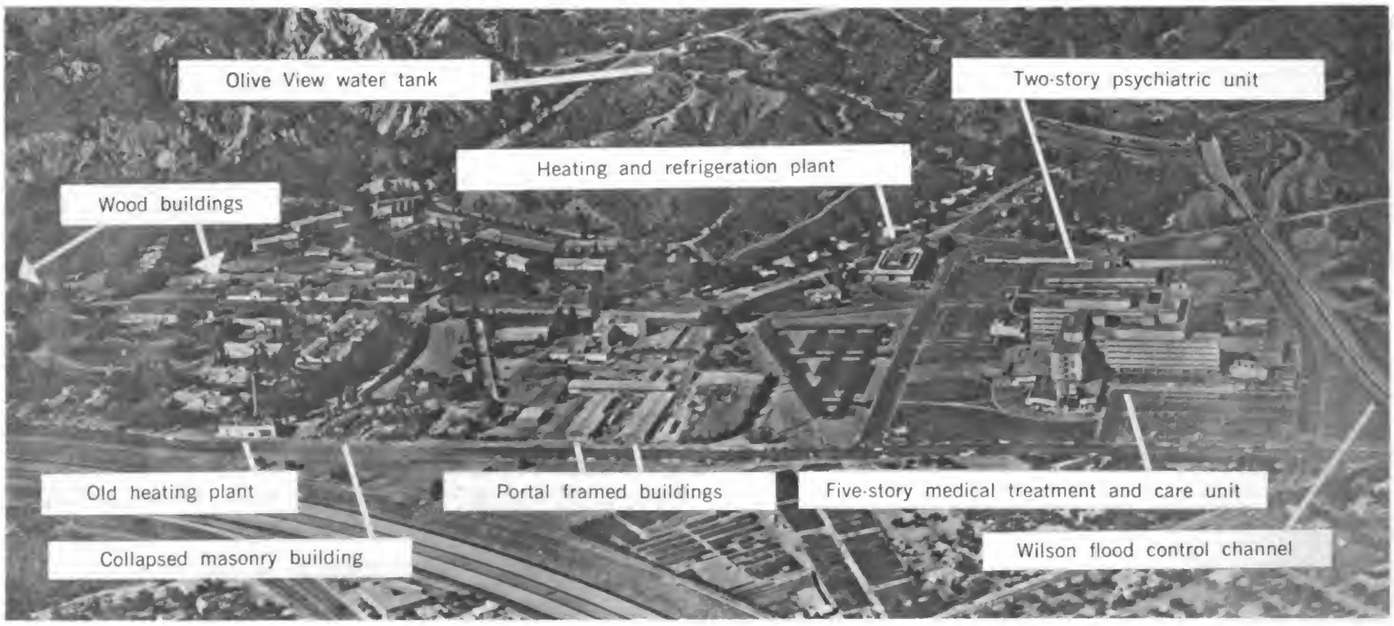

Figure 1.—Olive View Hospital complex, looking north. Old complex is to left and new complex to right. California Institute of Technology photograph.

been demolished. However, the wood frame residence-type structures were comparatively free from damage and continue to be occupied by hospital personnel.

Two one-story concrete portal framed buildings (fig. 1) were relatively undamaged. These were designed to resist lateral forces.

The severity of the ground motion is indicated by the damage that occurred in the underground tunnel system. In the utility pipe tunnel (fig. 5), which is 8 feet in height and covered with 7 feet of soil, the roof slab was displaced laterally, in relation to the floor slab, approximately 3 inches in the west portion and 1½ inches at the east end. In addition, there were many vertical cracks or separations approximately ½ inch in width observed throughout the length of the north-south tunnel.

This report presents a study only of the newer buildings (fig. 5), based on examination of the plans, specifications and computations, and an onsite inspection of the buildings. The structures to be reviewed are:

1 The Medical Care Facility (Building Report 24), including:
  a. Medical treatment and care unit
  b. Stair towers
  c. Warehouse
  d. Ambulance canopy
  e. Exhaust building
  f. Assembly building
  g. Walkway canopy
2 The psychiatric unit (Building Report 25)
3 The heating and refrigeration plant (Building Report 26)

Conclusions are made with each building as it is discussed. General recommendations for improving performance of all buildings are assembled at the end of this paper.

Certain conclusions and observations which have been reported by other engineers [1] and are now a part of the general record should be emphasized. They are as follows:

1 "The estimated ground motion, both vertical and horizontal, at this site greatly exceeded the specified code values and also produced stresses higher than the ultimate capacities of many structural members." The intensity of earthquake ground motion was probably in excess of 50 percent *g*.

2 "The lateral force design of the two (main) structures generally complied with the building code in existence at the time." The design and construction of these buildings were based on the Los Angeles County Building Code, 1965 edition, which is basically similar to the Uniform Building Code, 1964 edition.

[1] *Report on Olive View Hospital, San Fernando, Calif., Earthquake of February 9, 1971,* Structural Engineers Association of Southern California, Los Angeles, Calif., May 25, 1971.

Figure 2.—Olive View Hospital. New hospital complex, looking east. Note overturned stair towers at center and right. Los Angeles City Fire Department photograph.

Figure 3.—Olive View Hospital. Collapse of old portion of hospital convalescent wings. These buildings were not designed for lateral loads. U.S. Army Corps of Engineers photograph.

Figure 4.—Olive View Hospital. Damage to old powerplant of hospital. This building was not designed for lateral loads. U.S. Army Corps of Engineers photograph.

3 "From all available data, the grading and building codes related to engineering geology and soils engineering in effect at the time of the design were in compliance.

4 "No evidence was noted that construction procedures and quality of materials were not generally in accordance with plans and specifications."

MEDICAL CARE FACILITY

Medical Treatment and Care Unit

Description

This main unit is one structure and consists of four symmetrical five-story wings supported on a single, large one-story base. The wings are arranged to form a square around an open central core (figs. 6, 7, 8, 9, and 10). Because the surrounding ground slopes, the grade on the north and west sides of the building is level with the first floor, and the grade on the south and east elevations is one story lower at the ground-floor level. The base of the building, which extends beyond the configuration formed by the wings, is covered with earth and landscaping.

The basic framing scheme is a two-way flat slab reinforced concrete system supported either on tied or spiral columns (figs. 11,A and 24,H). The basement walls on the north and west sides, designed as cantilever retaining walls (fig. 11,C), are separated from the floor system by 4 inches. The foundations consist of spread footings founded on natural soils. All the concrete is regular stone concrete with a design strength of 3,000 psi, except the lower two-story concrete columns, which have a design strength of 5,000 psi. The reinforcing steel is deformed, 40,000 psi yield strength material, except the vertical bars in the columns which are 60,000 psi yield strength.

Stair structures at the ends of the wings, designed as free-standing cantilever towers, are separated structurally from the main building by 4 inches. The basic walls on three of the towers terminated at the first-floor level and were supported on a beam and column framing system. In the fourth (north) tower, the walls extend to the foundation pedestals.

Lateral Force System

The lateral forces were resisted by a system of shear walls above the second floor and moment-resisting concrete frames in the lower two stories. In effect, the scheme may be described as a four-story box structure supported on two levels of beam and column rigid frames. In accordance with code, a K value of 0.8 was used in the upper four stories because the concrete frames were designed to resist 25 percent of the lateral load. In the lower two stories, a K value of 0.67 was selected because the concrete frames were designed to resist 100 percent of the lateral load. A slightly greater base shear than required by code was calculated because the upper towers

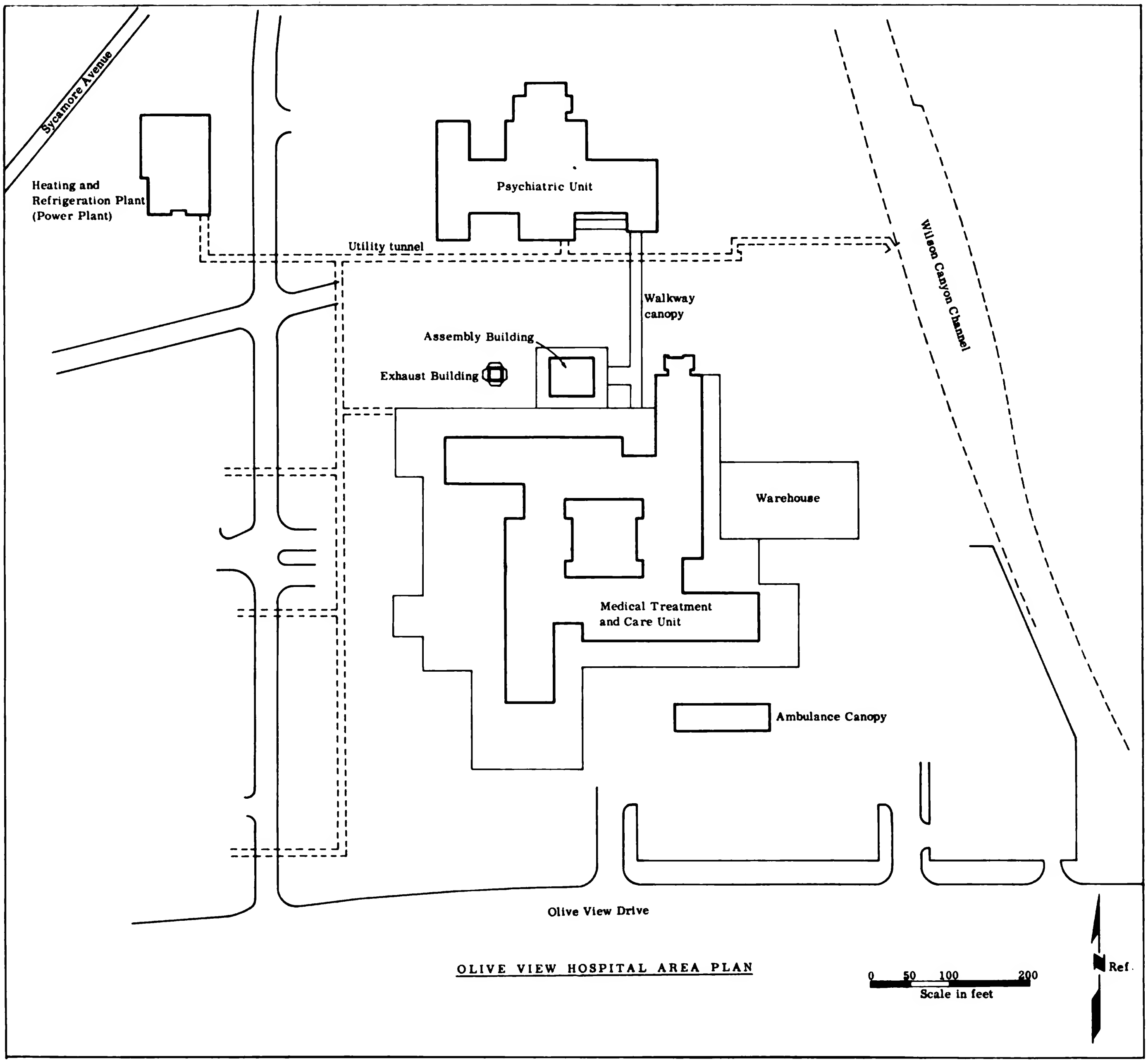

Figure 5.—Olive View Hospital. Newer buildings. Area plan of site, showing medical treatment and care unit, psychiatric unit, and heating and refrigeration plant.

were considered to form a "set-back" from the larger lower base structure. The lateral loads from the upper towers were applied as a concentrated load at the top of the base structure. The base structure was analyzed as a two-story structure with this concentrated load at the top. The base shear resulting from the 0.8 K factor used at the upper levels was not reduced to the level of the 0.67 K factor at the base level. This method resulted in a design shear force of 7.7 percent *g* above the second floor and 8.0 percent *g* at the ground level.

The stair towers were designed to resist lateral forces with reinforced concrete shear walls. The K factor was 1.0. The calculated base shear was 5.7 percent *g*. A reduction J factor, equal to 0.65, was applied to the calculated overturning forces.

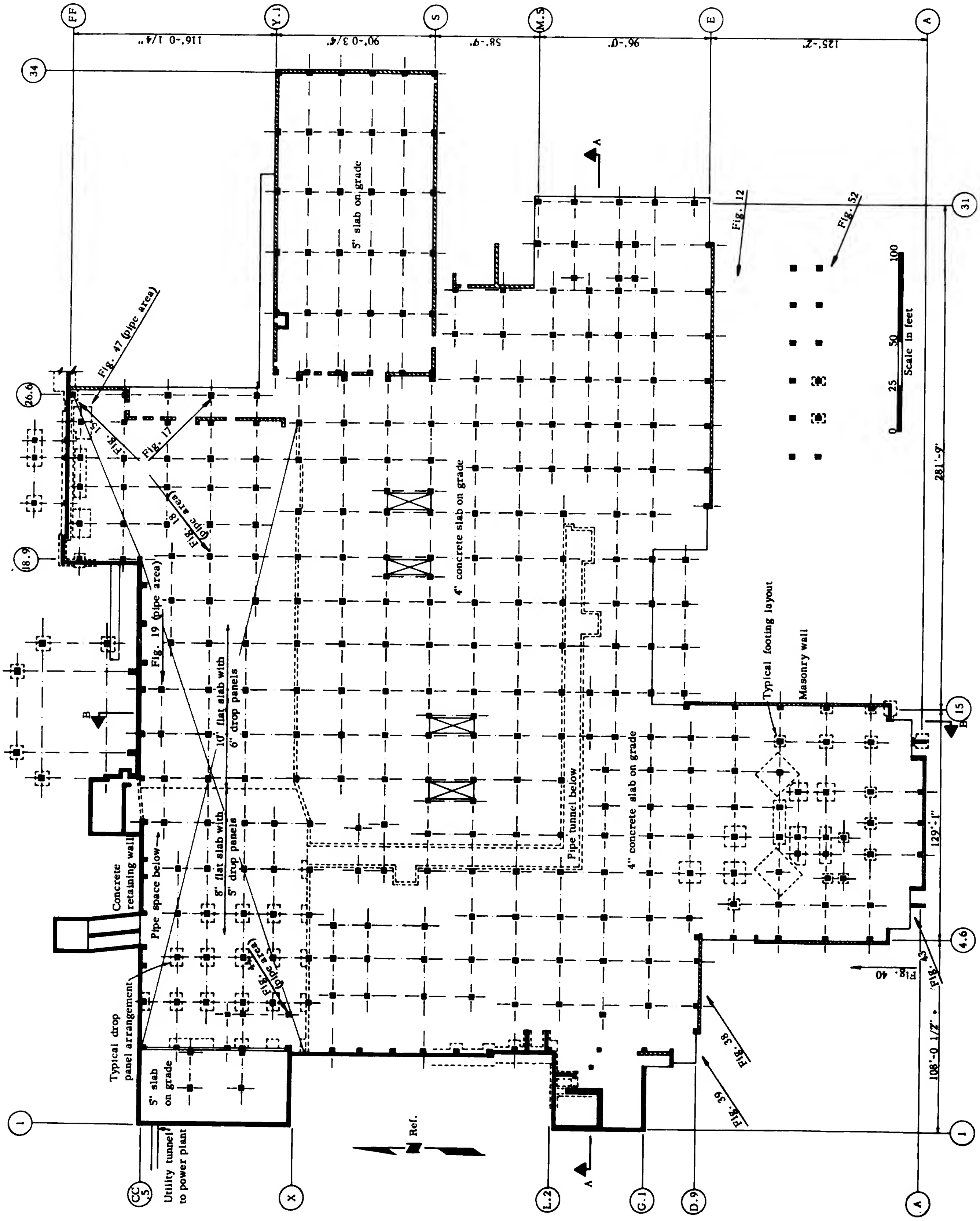

Figure 6.—Olive View Hospital, medical treatment and care unit, ground-floor and foundation plan.

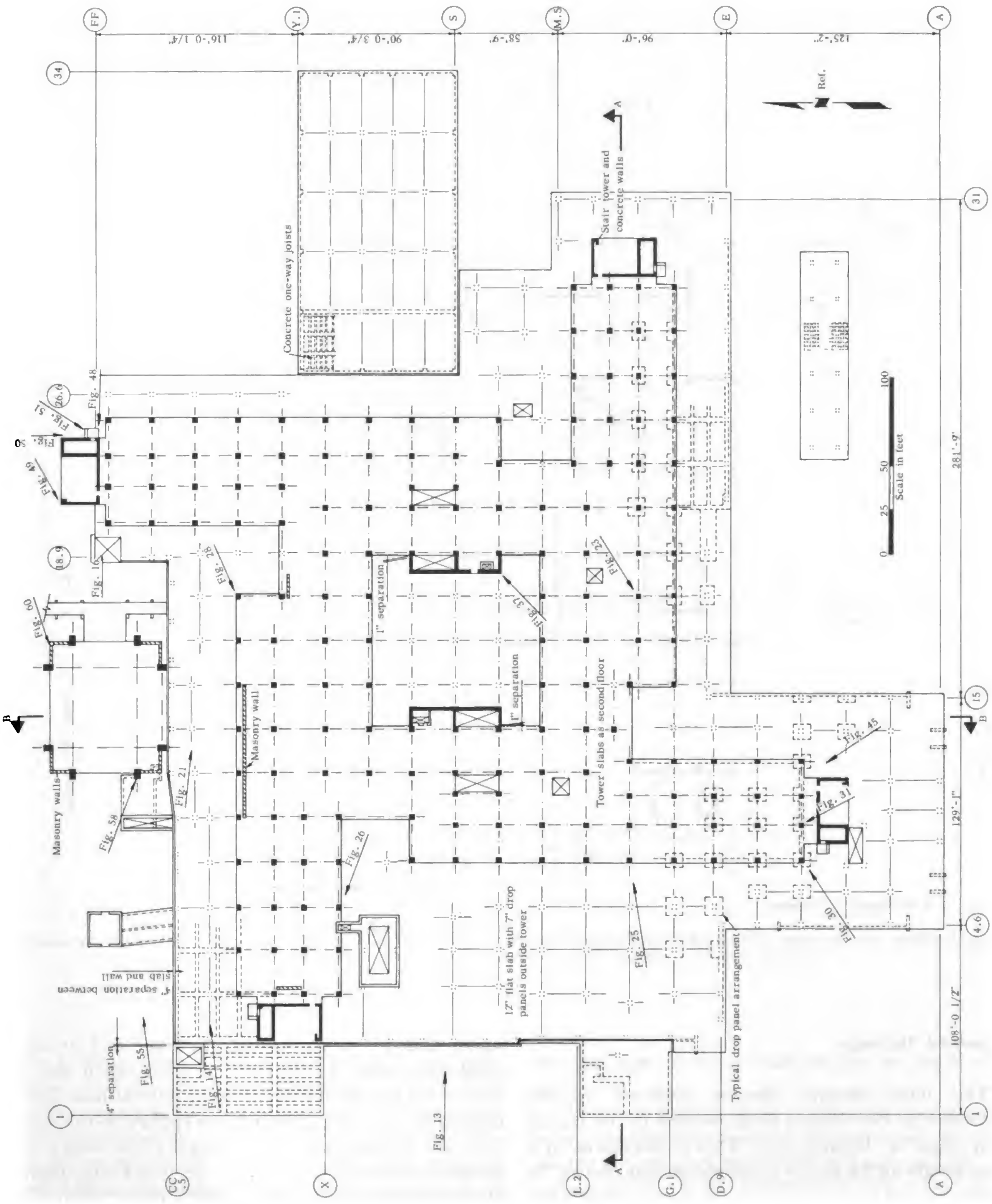

Figure 7.—Olive View Hospital, medical treatment and care unit. First-floor framing plan.

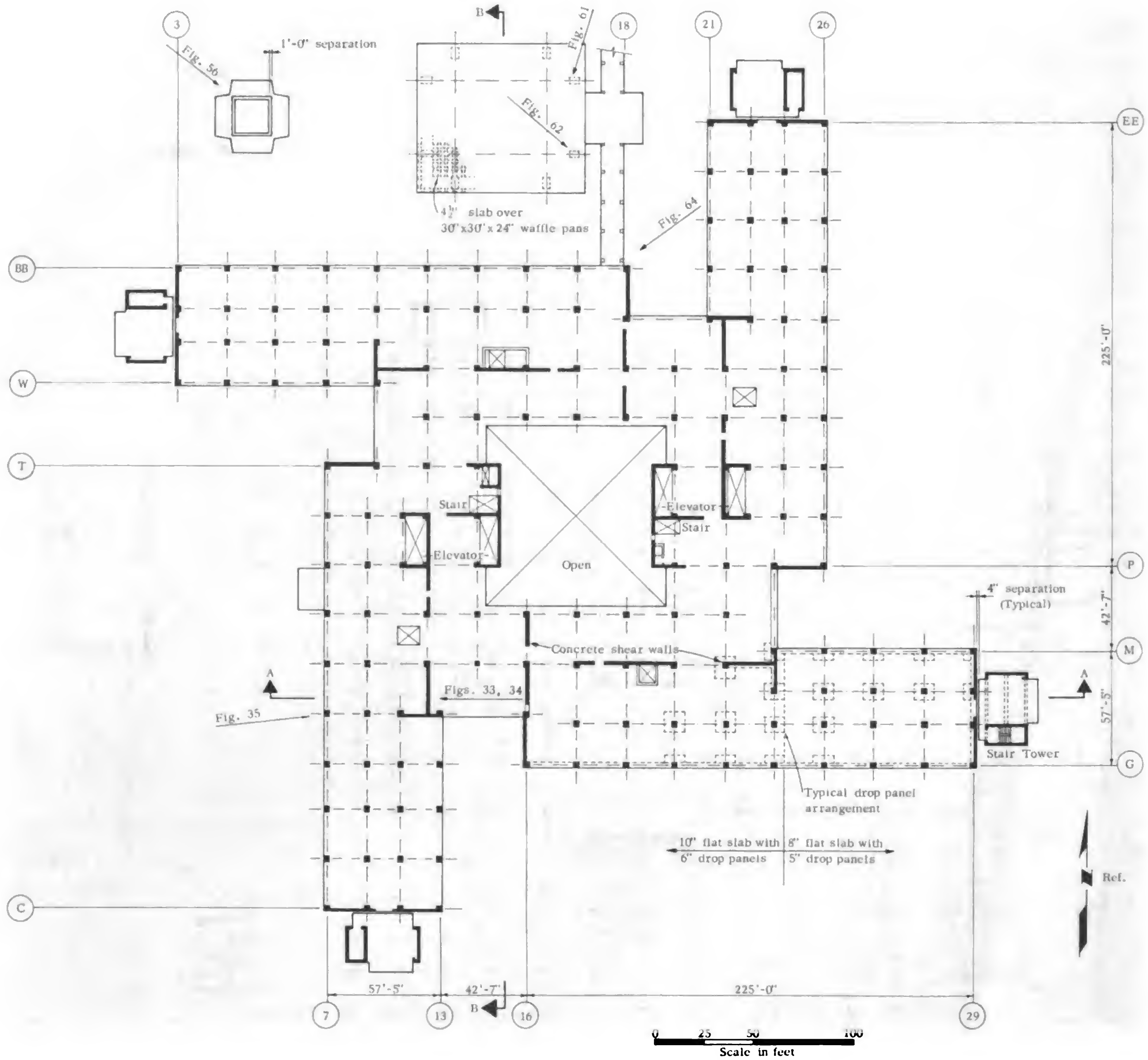

Figure 8.—Olive View Hospital, medical treatment and care unit. Second-floor framing plan. Third, fourth, and fifth floors are similar.

Observed Damage

The most obvious distress observed is the distortion in the column from the first to the second floor (figs. 12, 13, and 14). This displacement in a clear height of 14 feet is at the northwest corner, 28 inches toward the north and 15 inches toward the east; at the northeast corner, 15 inches toward the north and 15 inches toward the east; and at the southeast corner, 15 inches toward the north and a few inches toward the east. It is of interest that these distortions have not changed perceptibly in the year since the earthquake. To account for the difference in these movements, it should be noted that there are some differences in the stiffness characteristic of the various resisting frames, and further, that after

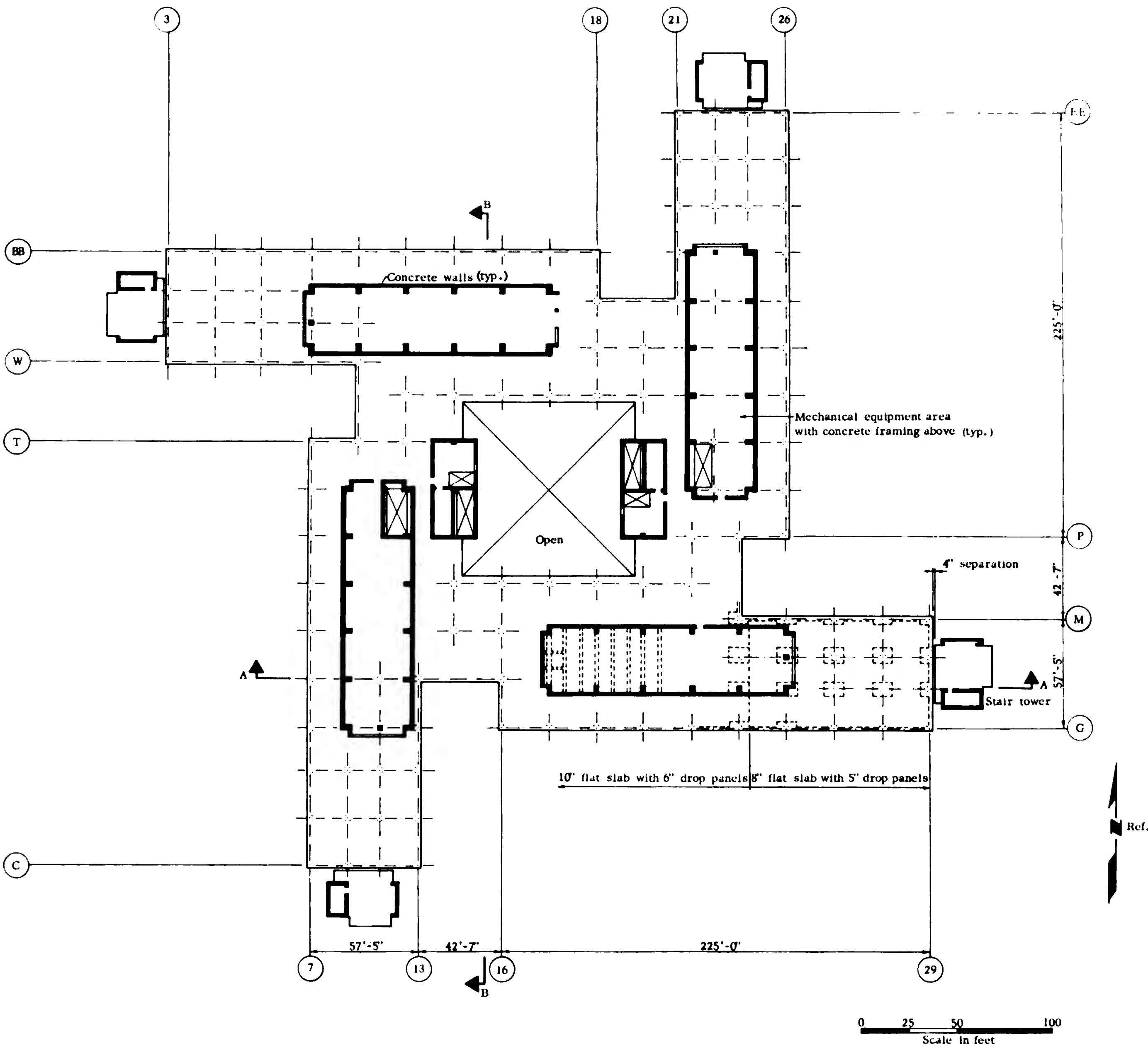

Figure 9.—Olive View Hospital, medical treatment and care unit. Roof framing plan.

the building moved, closing the minimal 4-inch separation between the resisting frames and the retaining walls, there was an uncalculated, yet effective, resistance developed by these walls. The pounding of the structure against these walls not only disturbed the soil behind the wall, but also caused tension cracks on the inside face (compression side) of the cantilever retaining walls (figs. 15 and 16). It is estimated that the tops of these walls moved as much as 6 inches.

There was failure of all of the tied columns located principally in the base structure (fig. 17) outside the area supporting the four five-story towers.

For example, in the area to the north of the tower structure, many and various types of failures in structural members were observed. The tied col-

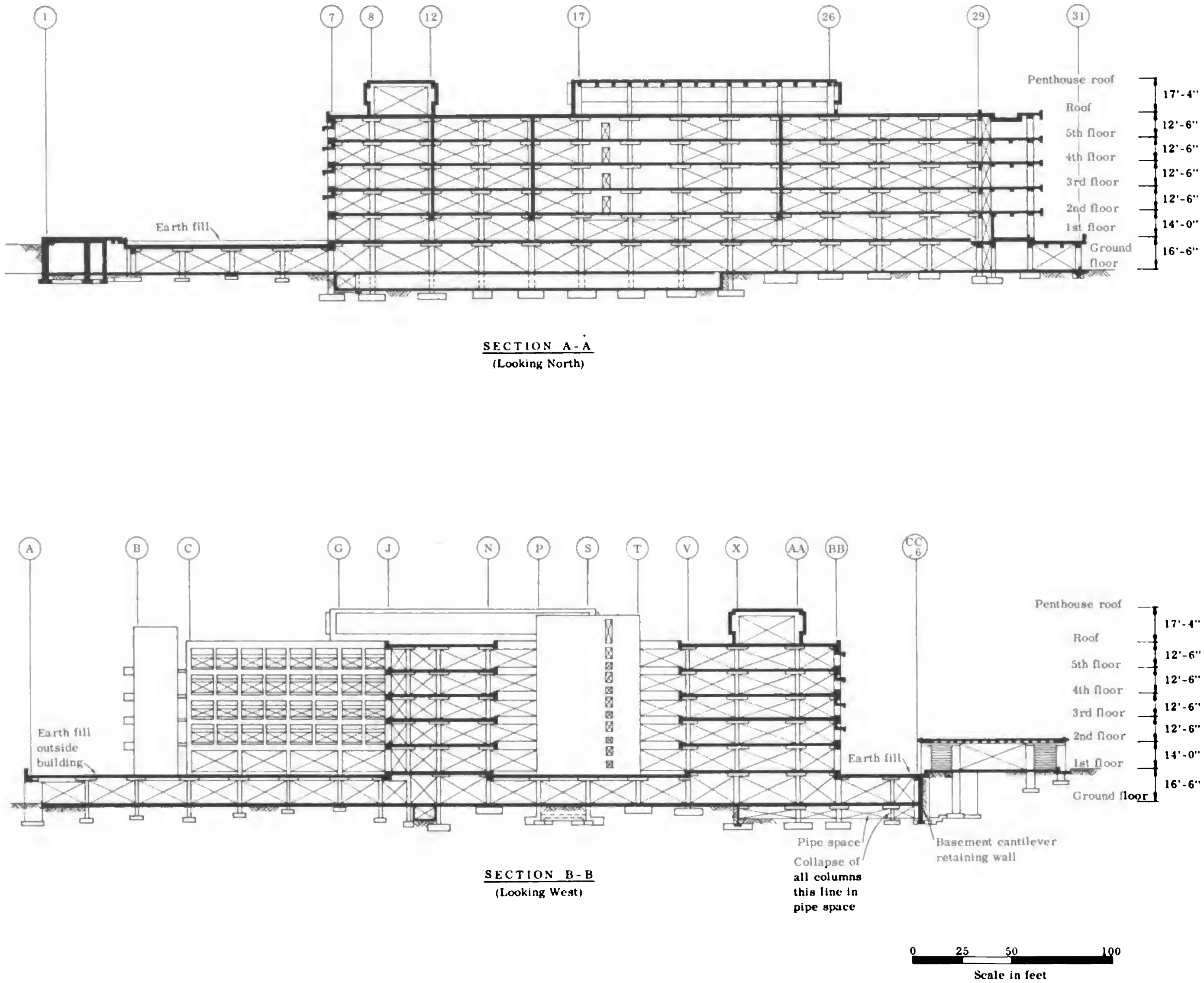

Figure 10.—Olive View Hospital, medical treatment and care unit. Building cross sections.

umns located in the pipe space near the north basement wall failed (fig. 18); a continuous crack extending east and west occurred in the ground-floor flat slab (fig. 19); a similar-type crack in the beam and slab framing of the first floor also was apparent (figs. 20 and 21). It was evident that all of these members had been subjected to very high shear stresses, and that either the reinforcing steel was insufficient to resist the magnitude of the forces or the reinforcing steel was mislocated or ineffectively spaced to withstand the unpredictable forces generated in the continuous framing. Further evidence of this type of damage occurred in the flat slab drop panels at the column supports. The cracking and spalling of the concrete (figs. 22 and 23) are also quite general throughout the first and second floors.

In the first story, the rigid frames that were most effective in resisting the major portion of the lateral loads were located along the perimeter walls of the towers. Significantly, the tops of the vertical column members were restrained (fixed) by the beams and walls supporting the towers. In most instances during the earthquake, the horizontal beam framing and the column members were capable of resisting stresses from excessive lateral deformations, because the longitudinal reinforcing steel was continuous and the closely spaced shear reinforcing provided good confinement of the concrete (fig. 24,D). These

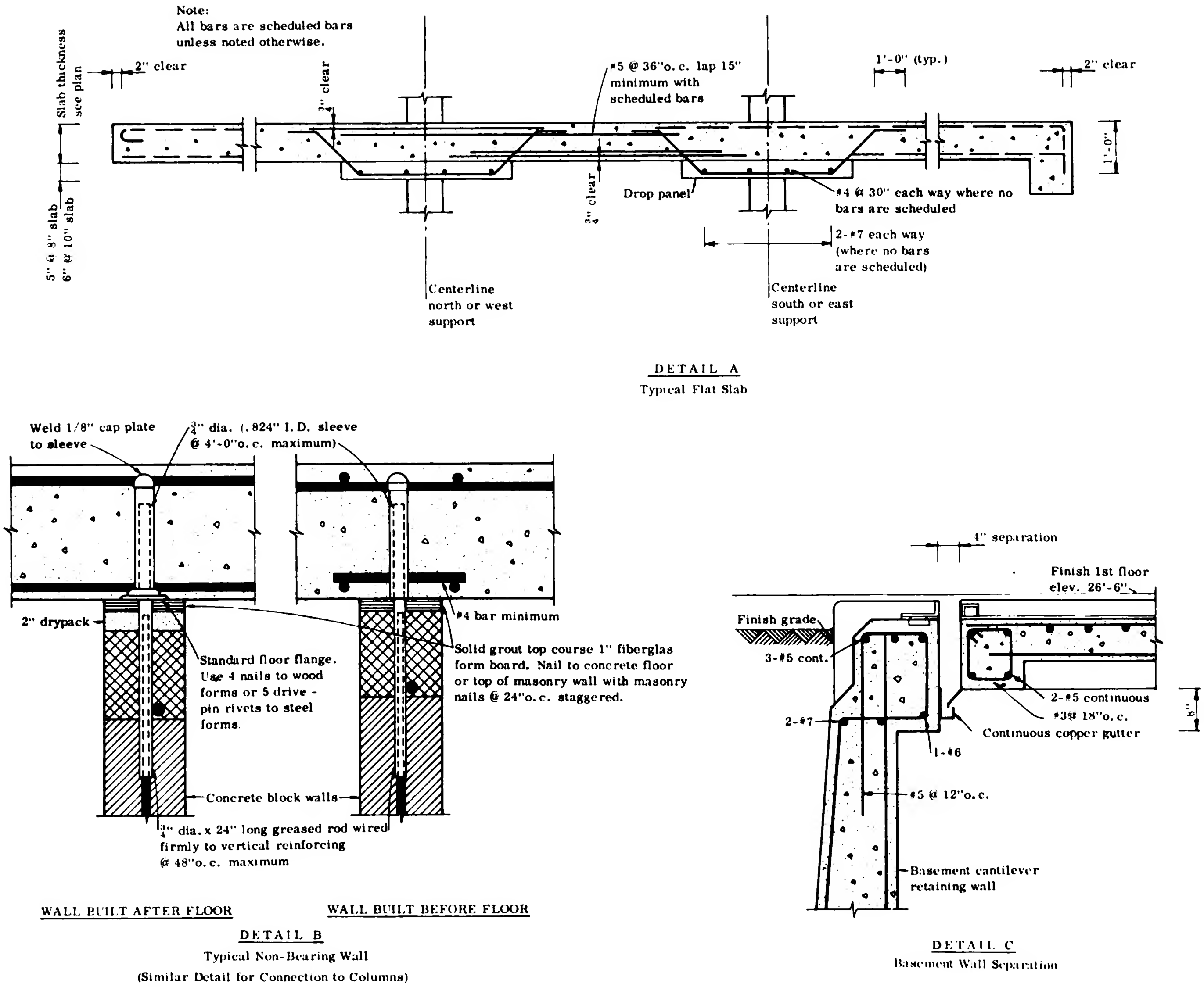

Figure 11.—Olive View Hospital, medical treatment and care unit. Typical details of flat slab, nonbearing wall, and basement wall separaton at first floor.

characteristics also were present in the columns with closely spaced spiral reinforcing (figs. 24,E and 25). There was distress in many of the beam-column connections (figs. 26 and 27) where the stirrups or ties did not extend through the joints. Failure occurred in the corner tied columns (figs. 24,E, 28, and 29). Interestingly, the vertical and lateral loads, which caused the collapse of these corner columns, were transferred to the adjacent exterior columns through the cantilever action of the walls above the column (figs. 30 and 31).

Above the second story, the lateral loads were resisted by the interior concrete shear walls (figs. 32 and 33) and the exterior concrete piers between the windows (figs. 24,F, 35, and 36). These members were stressed to capacity in shear. Their shear capacity was approximately twice that resulting from code forces.

In addition to observed structural damage, there was considerable distress in the nonstructural features. In summary, nonstructural masonry walls were torn loose (figs. 11,B and 37); many precast concrete fascia elements were dislodged (figs. 38 and 39); walls collapsed (fig. 40); connections anchoring the precast concrete failed (figs. 41 and 42); interior partitions, ceilings, and other architectural features were damaged severely (fig. 43); and mechanical and electrical equipment failed to function.

Figure 12.—Olive View Hospital, medical treatment and care unit. View of south side, looking west, from ground-floor level. Note collapsed ground-floor roof, which is covered by planting, and loss of precast fascia panels from roof spandrel.

Figure 13.—Olive View Hospital, medical treatment and care unit. View of west side at main entrance from first-floor level.

Figure 14.—Olive View Hospital, medical treatment and care unit. View of north side, looking east, from first-floor level. Floor slope is result of collapsed columns below and is emphasized by walkway canopy in background.

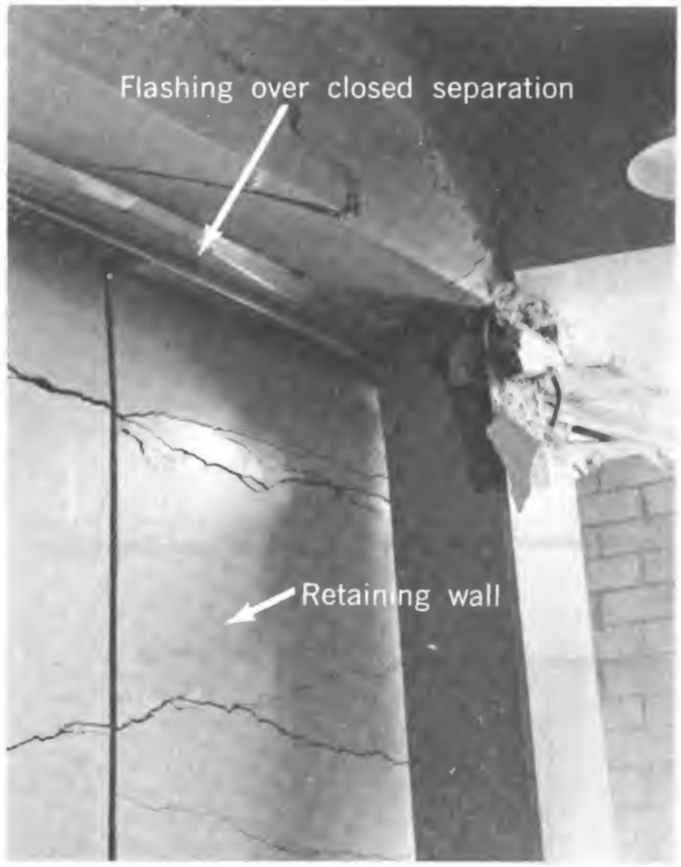

Figure 15.—Olive View Hospital, medical treatment and care unit. Horizontal cracks in north basement wall in ground-floor story. Note closure of separation at top of wall.

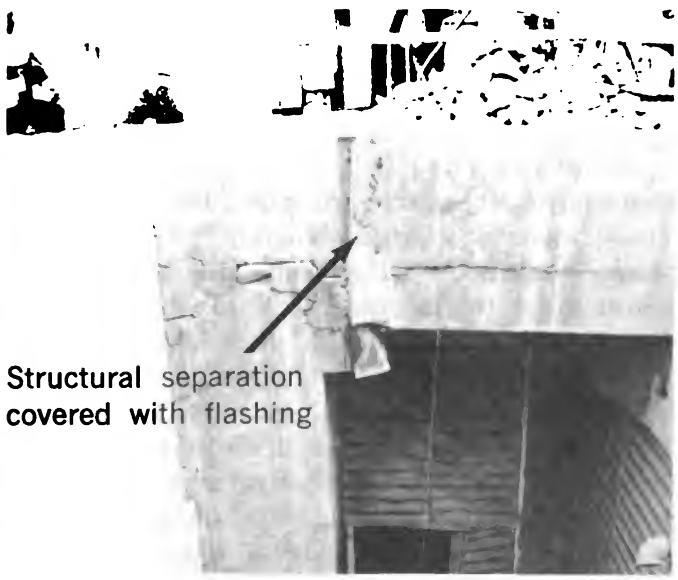

Figure 16.—Olive View Hospital, medical treatment and care unit. Areaway pier at northeast wing, showing effects of hammering by first-floor framing through wall separation.

Figure 17.—Olive View Hospital, medical treatment and care unit. Complete collapse of tied column located at ground-floor level loading dock at east side.

Figure 18.—Olive View Hospital, medical treatment and care unit. Complete collapse of tied column located in pipe space area near north wall.

Figure 19.—Olive View Hospital, medical treatment and care unit. Continuous crack formed in underside of ground-floor framing near north wall. Crack is result of collapse of tied columns.

Figure 20.—Olive View Hospital, medical treatment and care unit. Closeup view of figure 19, showing lack of bottom reinforcing bars through crack.

Figure 21.—Olive View Hospital, medical treatment and care unit. Crack formed under first floor framing near a column near the north wall, where column has collapsed below in pipe space.

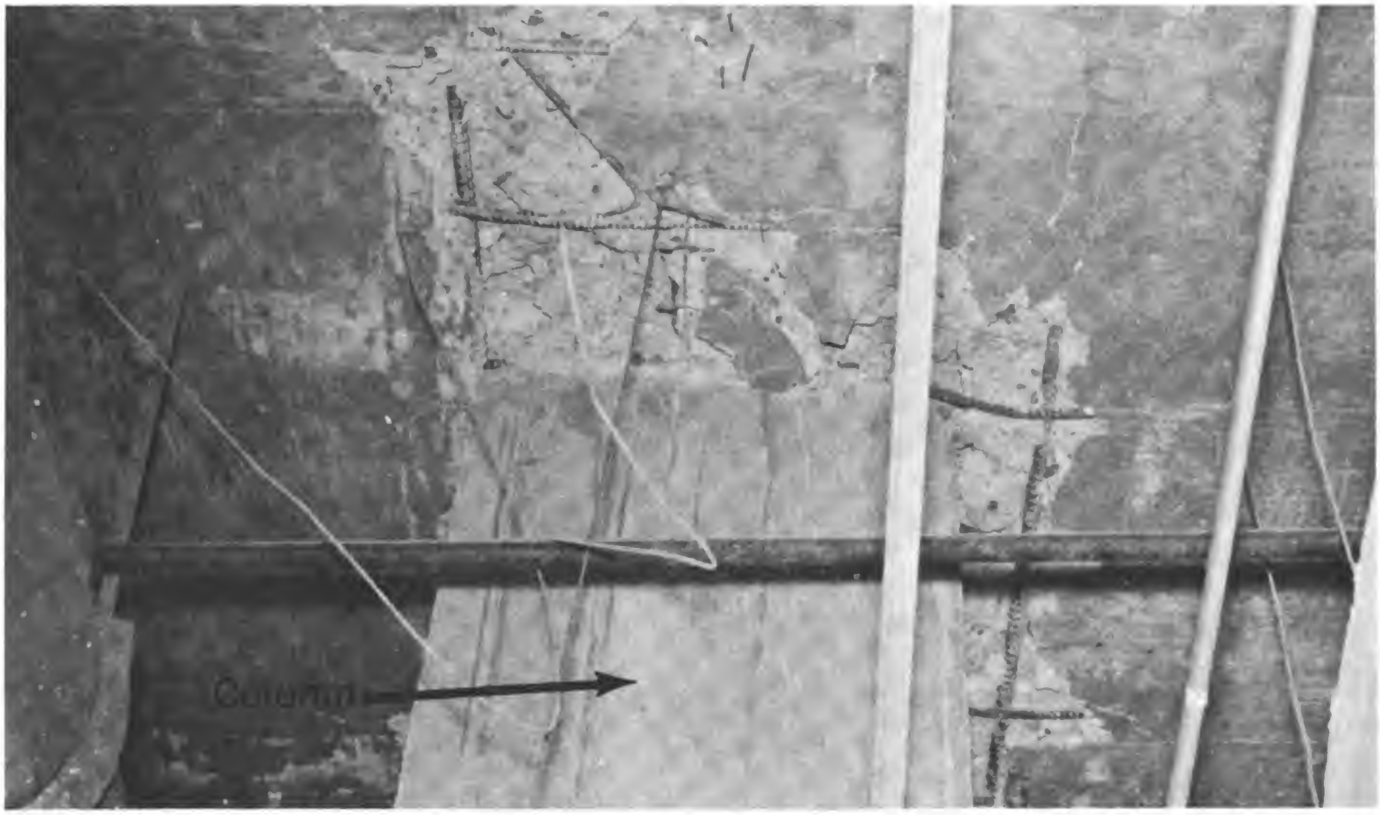

Figure 22.—Olive View Hospital, medical treatment and care unit. Spalling of concrete slab under second-floor framing caused by rotation through connection.

Figure 23.—Olive View Hospital, medical treatment and care unit. Distortion of column (shown in fig. 22) at first floor.

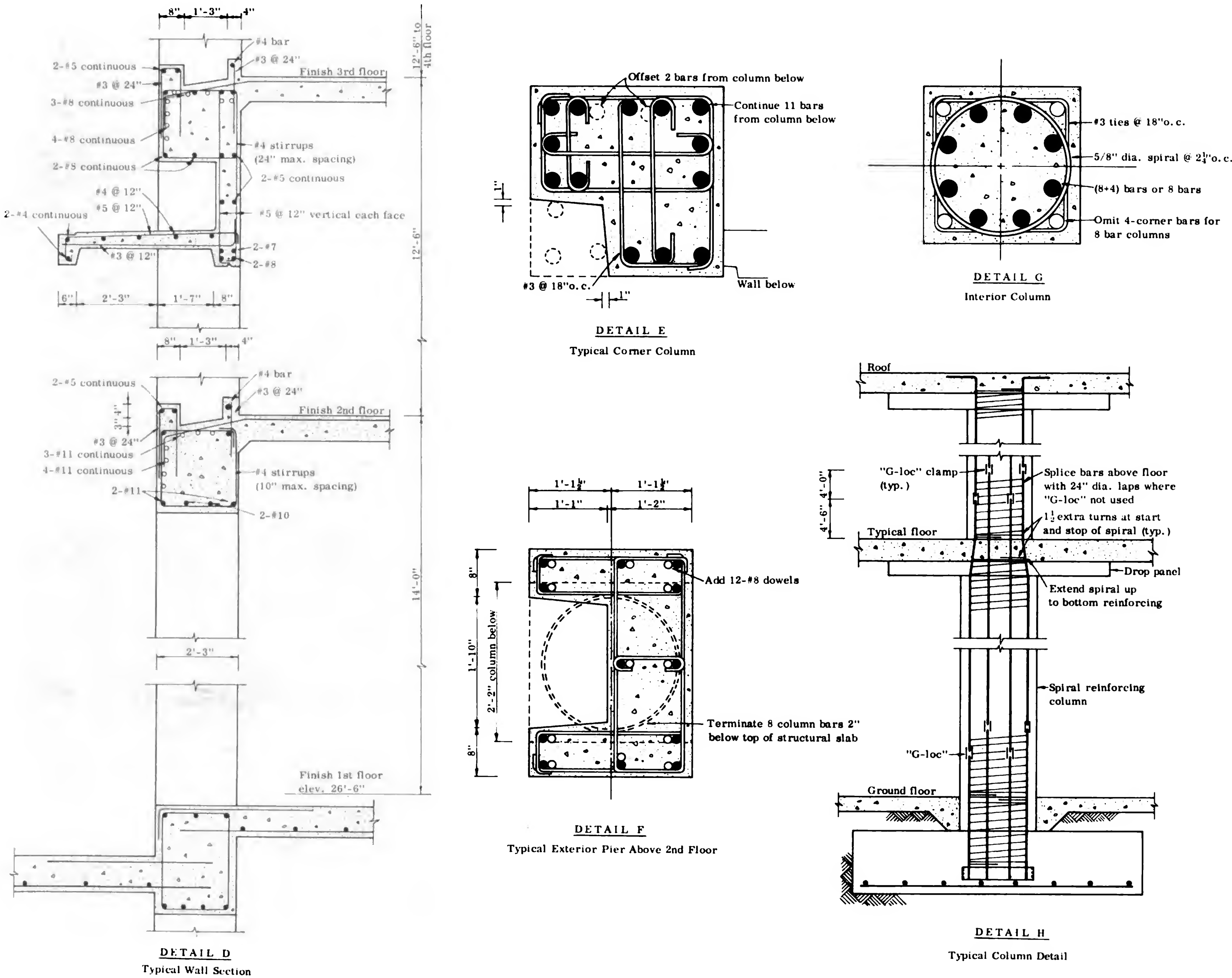

Figure 24.—Olive View Hospital, medical treatment and care unit. Wall and column details.

Figure 25.—Olive View Hospital, medical treatment and care unit. Distortion of exterior column at first-floor level.

Figure 27.—Olive View Hospital, medical treatment and care unit. Beam-to-column connection detail of figure 26. Note lack of confinement to beam reinforcing bars in this area. Bottom bars also were lapped in this area.

Figure 26.—Olive View Hospital, medical treatment and care unit. Distortion of exterior column at first-floor level.

Figure 28.—Olive View Hospital, medical treatment and care unit. Collapse of corner column at first-floor level.

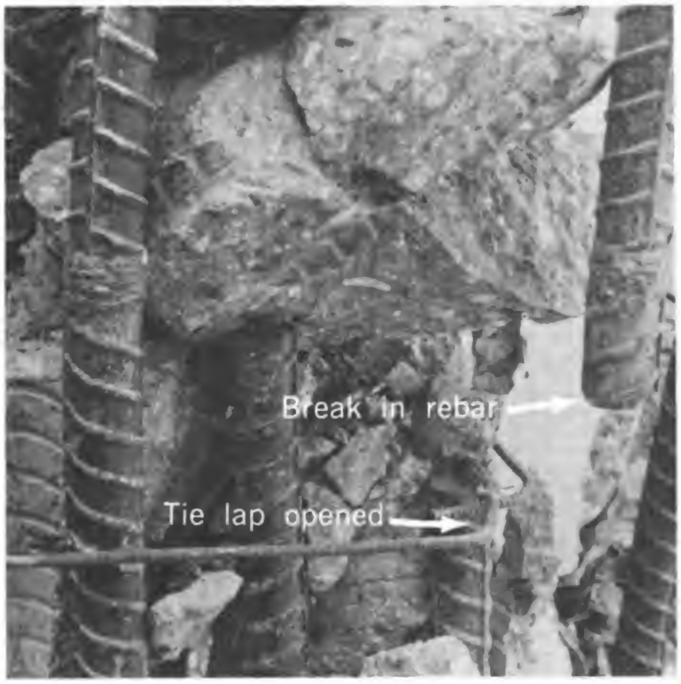

Figure 29.—Olive View Hospital, medical treatment and care unit. Fracture detail of reinforcing bars near a full-penetration butt weld for column in figure 28. Note opening of column tie laps at corner.

Figure 30.—Olive View Hospital, medical treatment and care unit. View at end of building wing at first-floor level. The wall above probably has carried collapsed corner column load to interior column line. Stair tower is at right of photo.

Figure 31.—Olive View Hospital, medical treatment and care unit. Detail view of adjacent column of figure 30, showing distress to connection where confinement was not provided.

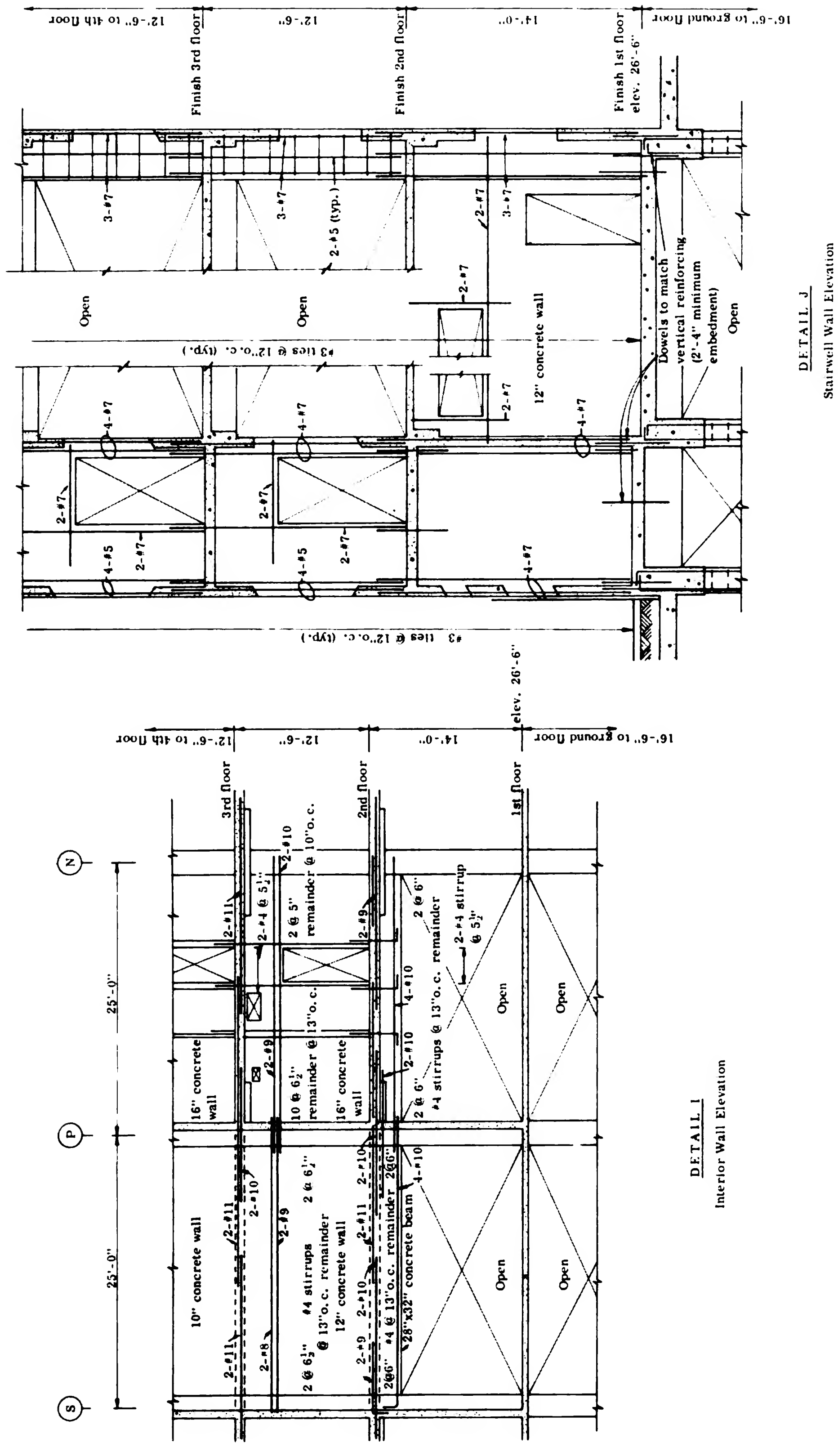

Figure 32.—Olive View Hospital, medical treatment and care unit. Typical interior wall and stairwell wall elevation details.

Figure 33.—Olive View Hospital, medical treatment and care unit. Diagonal cracks in shear wall at second floor.

Figure 34.—Olive View Hospital, medical treatment and care unit. Diagonal cracks and spalling at floorline at wall shown in figure 33.

Figure 35.—Olive View Hospital, medical treatment and care unit. Diagonal shear cracks in second-floor exterior wall panels viewed from outside.

Figure 36.—Olive View Hospital, medical treatment and care unit. Diagonal cracks in figure 35 viewed from inside.

Figure 37.—Olive View Hospital, medical treatment and care unit. Shift of southwest corner of east elevator at first-floor level. Wall was designed to be nonparticipating in resisting lateral loads and was separated from floor framing at this level.

Figure 40.—Olive View Hospital, medical treatment and care unit. Masonry veneer walls lying on ground-floor level. These fell away from building due to earthquake movements.

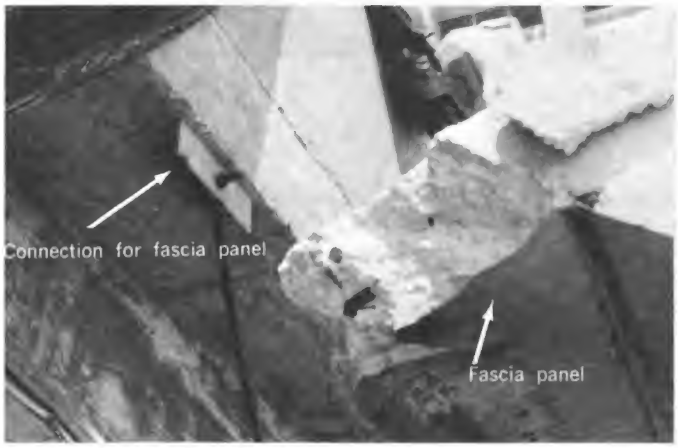

Figure 38.—Olive View Hospital, medical treatment and care unit. View from below of precast fascia panel connection to the concrete beams along first floor. View is at southwest corner of building.

Figure 41.—Olive View Hospital, medical treatment and care unit. Upper connection of precast unit in figure 38 where failure occurred in precast unit concrete.

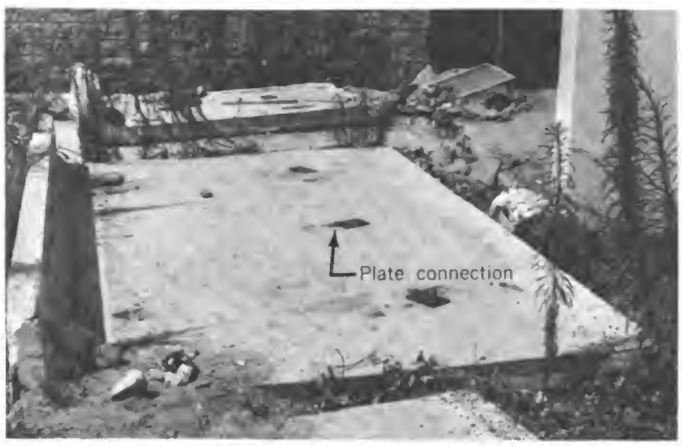

Figure 39.—Olive View Hospital, medical treatment and care unit. Precast units lying on ground. These were supported at first-floor level before earthquake.

Figure 42.—Olive View Hospital, medical treatment and care unit. Lower connection of precast panel. Plate insert cast into panel was welded to stud bolts shown in figure 38.

Figure 43.—Olive View Hospital, medical treatment and care unit. Ground-floor level framing at southwest corner which supports first-floor planting on roof. Note loss of plastered soffit and window wall damage from excess movement.

Figure 44.—Olive View Hospital, medical treatment and care unit. Distress to ground-floor flat slab drop panel adjacent to north wall. Some indication of uplift of north stair tower to retaining wall was indicated by splitting of drop panel area.

Figure 45.—Olive View Hospital, medical treatment and care unit. Tilt of north stair tower from building viewed from ground-floor level.

Figure 46.—Olive View Hospital, medical treatment and care unit. Northwest corner of north stair tower below first-floor ground level. Note complete collapse of tied concrete column below grade beam. LeRoy Crandall & Associates photograph.

Figure 48.—Olive View Hospital, medical treatment and care unit. Southeast connection of north stair tower to basement retaining wall at left side of photo. LeRoy Crandall & Associates photograph.

Figure 47.—Olive View Hospital, medical treatment and care unit. Northeast corner of north stair tower with close view of column collapse. LeRoy Crandall & Associates photograph.

Figure 49.—Olive View Hospital, medical treatment and care unit. Collapse of southeast column of west stair tower in pipe space below ground floor. Tied column was separated from ground-floor slab in design.

Figure 50.—Olive View Hospital, medical treatment and care unit. South end view of south wing with stair tower in its overturned position.

Figure 51.—Olive View Hospital, medical treatment and care unit. Bottom view of south stair tower with first-floor framing shown intact. Note the S-shaped column bars which extended to foundation.

Figure 52.—Olive View Hospital, ambulance canopy. View looking north, showing collapse of north tied columns and subsequent rotation at south column line.

Figure 53.—Olive View Hospital, ambulance canopy. Closeup view of south column upper connection to roof framing.

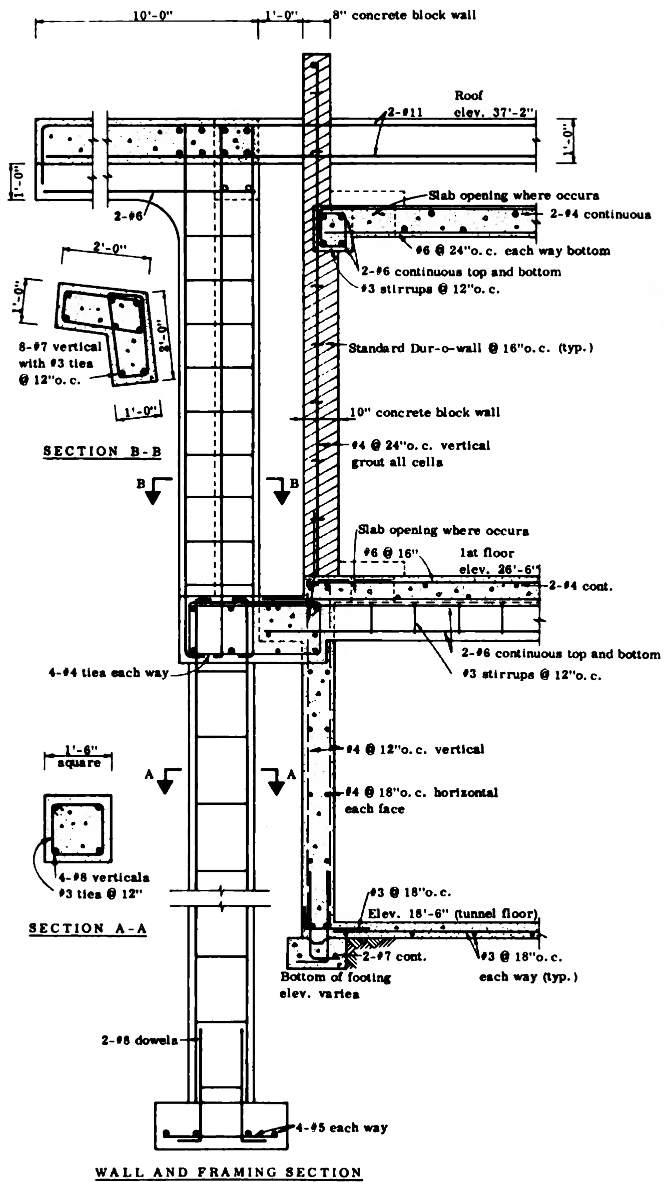

Figure 54.—Olive View Hospital, exhaust building. Section details through building wall and framing.

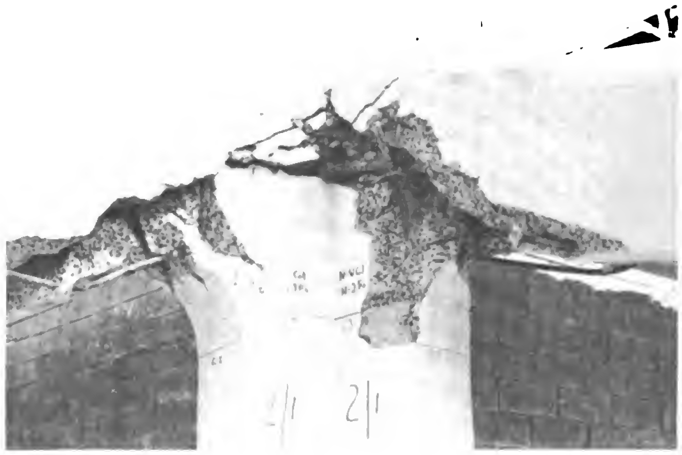

Figure 56.—Olive View Hospital, exhaust building. View of roof-to-column connection, indicating the loss of resistance due to lack of anchorage for column reinforcing bars.

Figure 57.—Olive View Hospital, exhaust building. View on inside face of figure 56.

Figure 55.—Olive View Hospital, exhaust building. View looking northeast, with assembly building and north stair tower in background. Note ground uplift (at right of photo) as result of basement wall movement.

Figure 58.—Olive View Hospital, assembly building. Southwest corner vertical cantilever column supporting roof framing. Column is fixed below the first-floor ground level and is connected at roof as shown in figure 59.

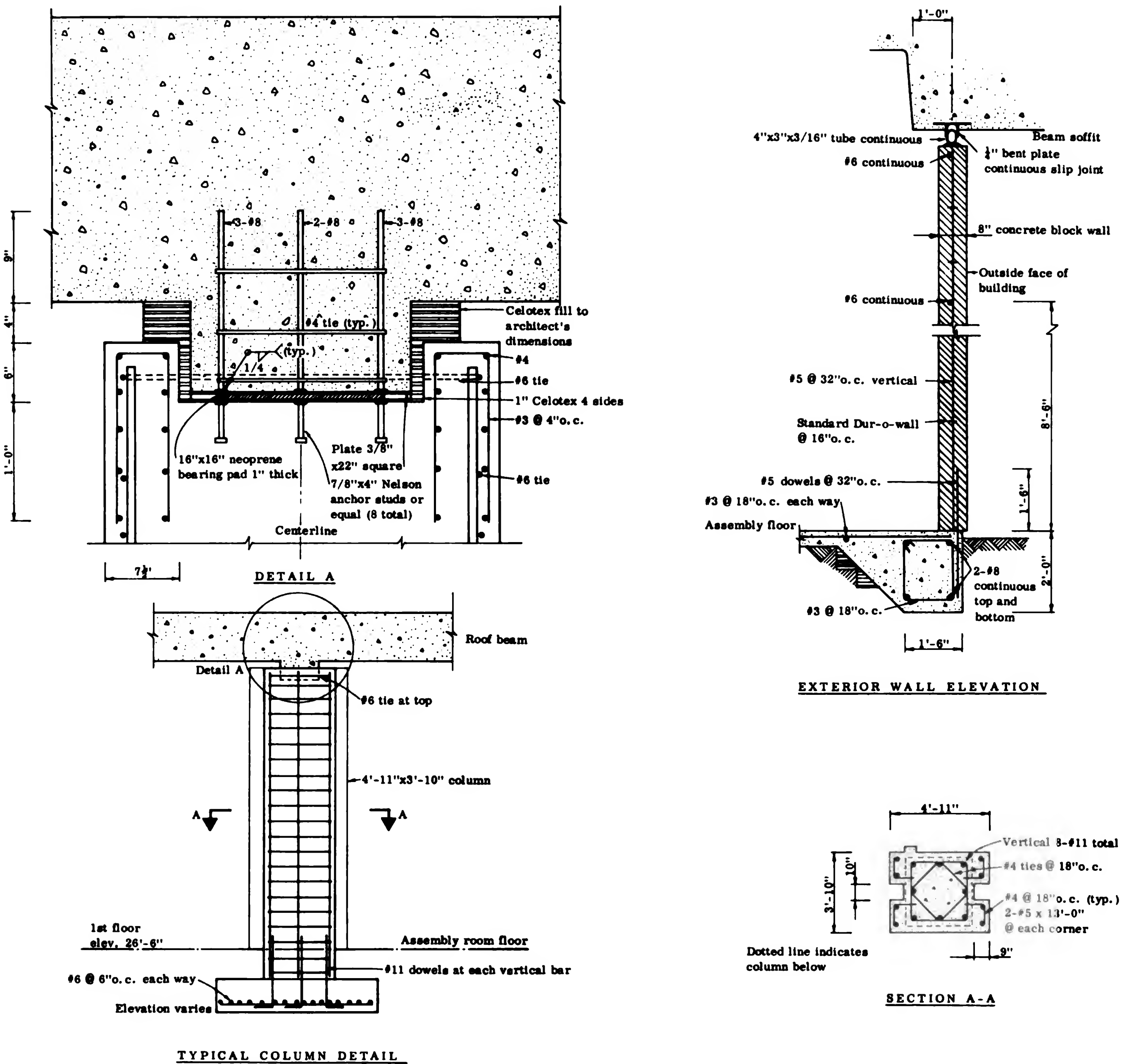

Figure 59.—Olive View Hospital, assembly building. Details of exterior column and masonry wall connections.

Figure 60.—Olive View Hospital, assembly building. Distortion and cracking of nonbearing masonry walls at northeast corner. Note damage of roof-to-column connection.

Figure 61.—Olive View Hospital, assembly building. Closeup view of recessed roof connection to column. Note bearing pads.

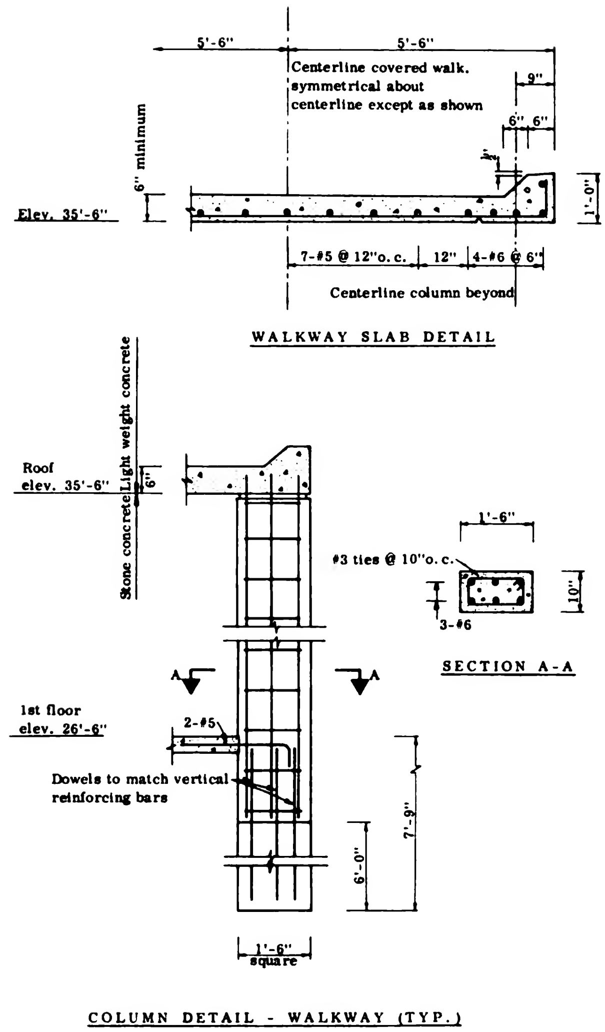

Figure 63.—Olive View Hospital, walkway canopy. Details of column and roof connections.

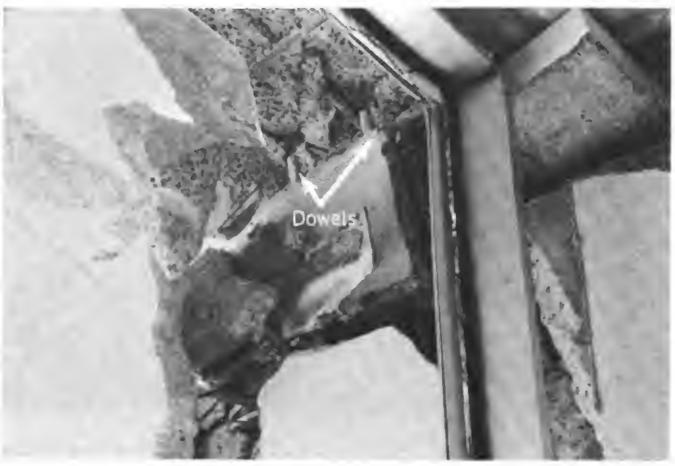

Figure 62.—Olive View Hospital, assembly building. Closeup view of recessed roof connection, showing failure of dowels into roof slab due to lack of anchorage.

Figure 64.—Olive View Hospital, walkway canopy. View of south end where the cantilevered roof cracked due to movement between medical and psychiatric buildings.

Figure 65.—Olive View Hospital, walkway canopy. Collapse of cantilevered roof, resulting from collapse of psychiatric unit.

Figure 66.—Olive View Hospital, walkway canopy. Separation of roof slab from top of column. Note lack of anchorage of column bars into slab.

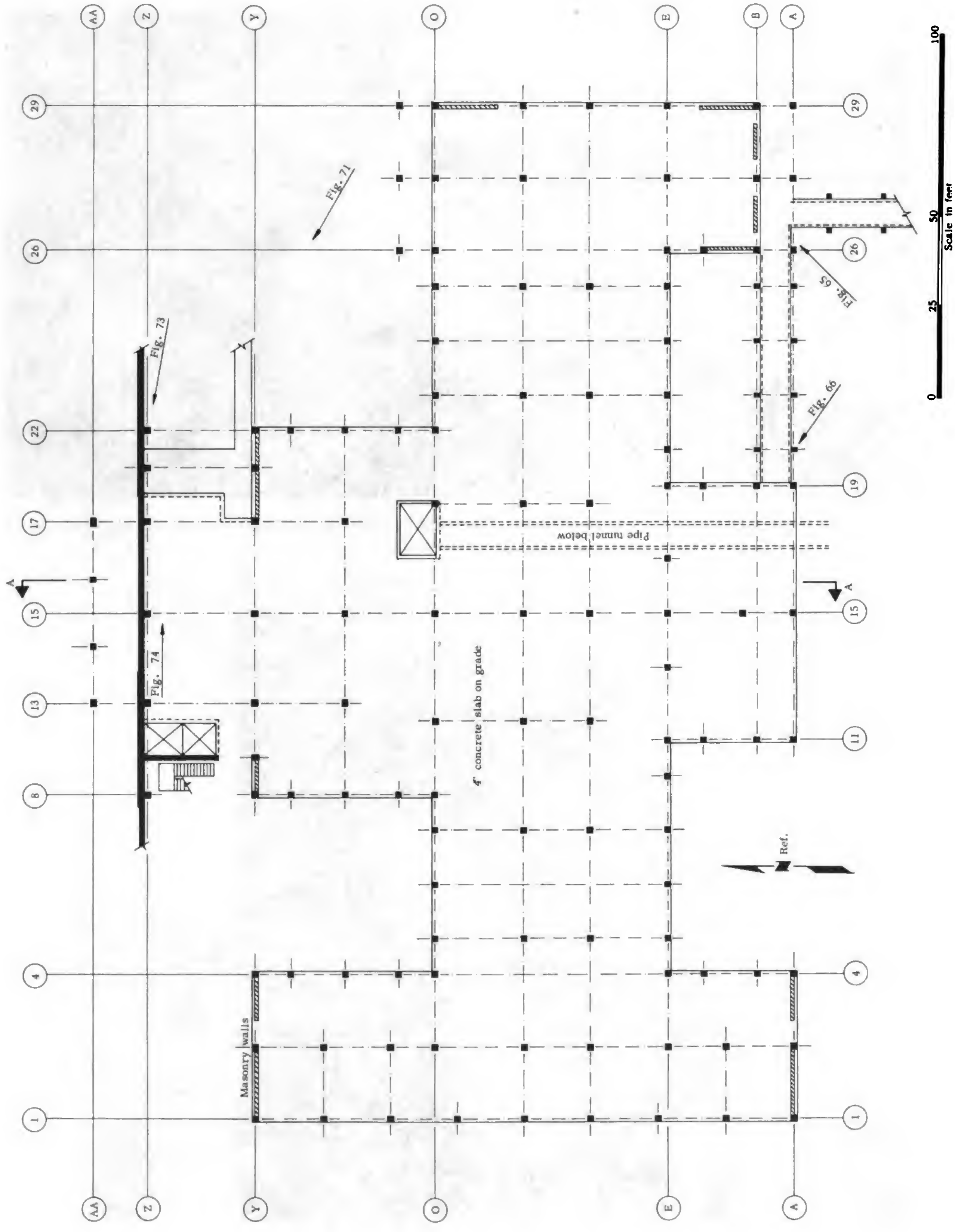

Figure 67.—Olive View Hospital, psychiatric unit. First-floor and foundation plan.

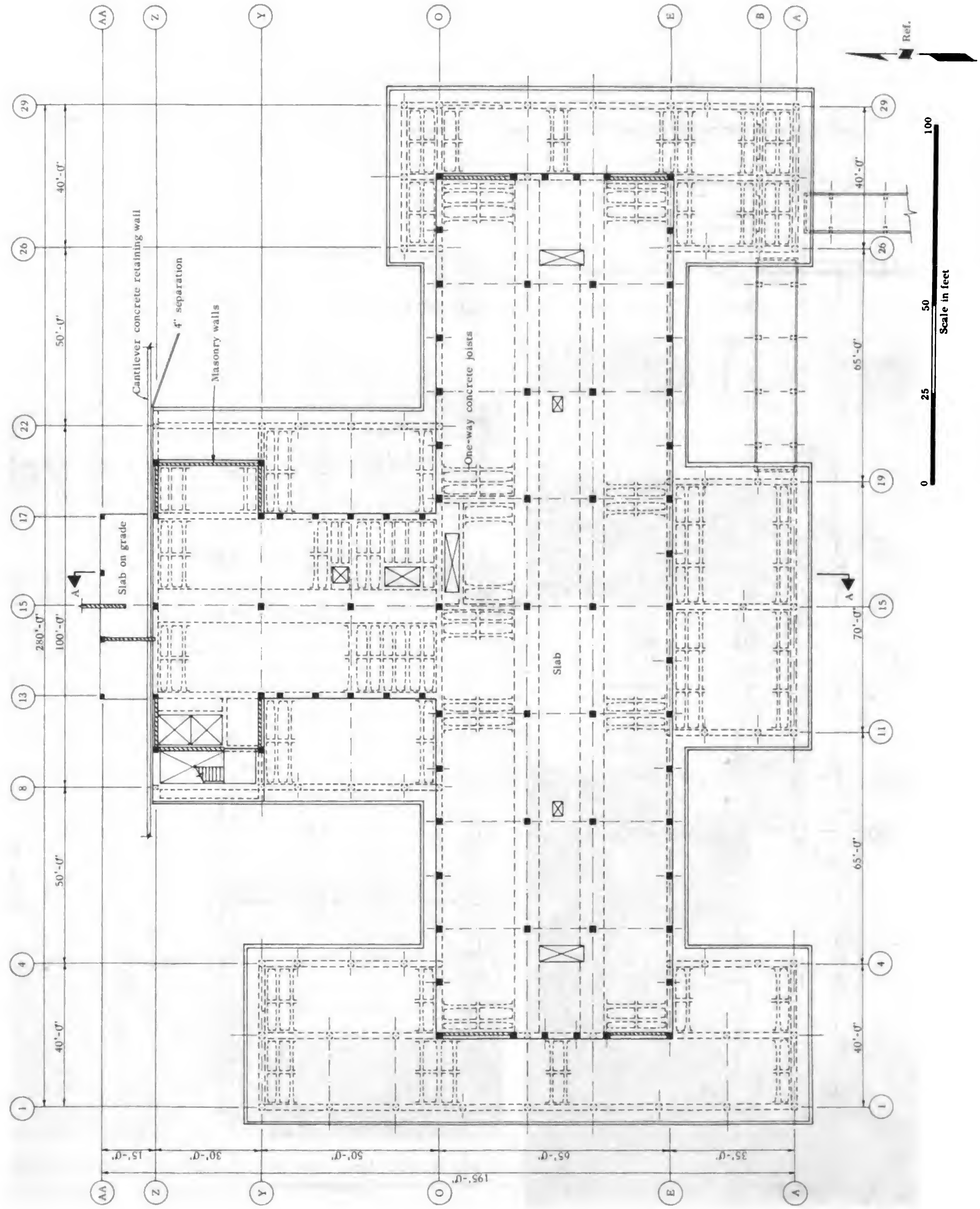

Figure 68.—Olive View Hospital, psychiatric unit. Second-floor framing plan.

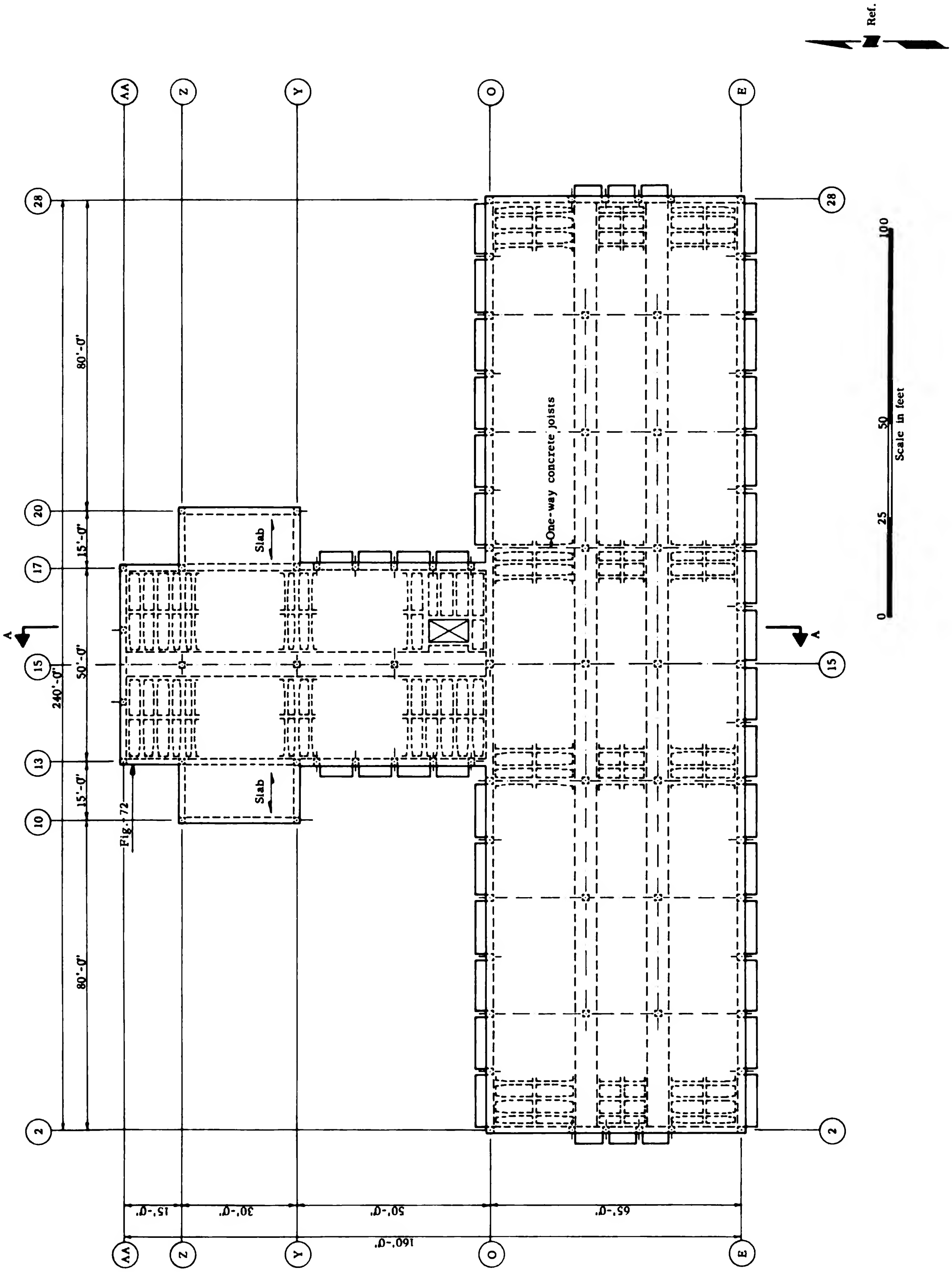

Figure 69.—Olive View Hospital, psychiatric unit. Roof framing plan.

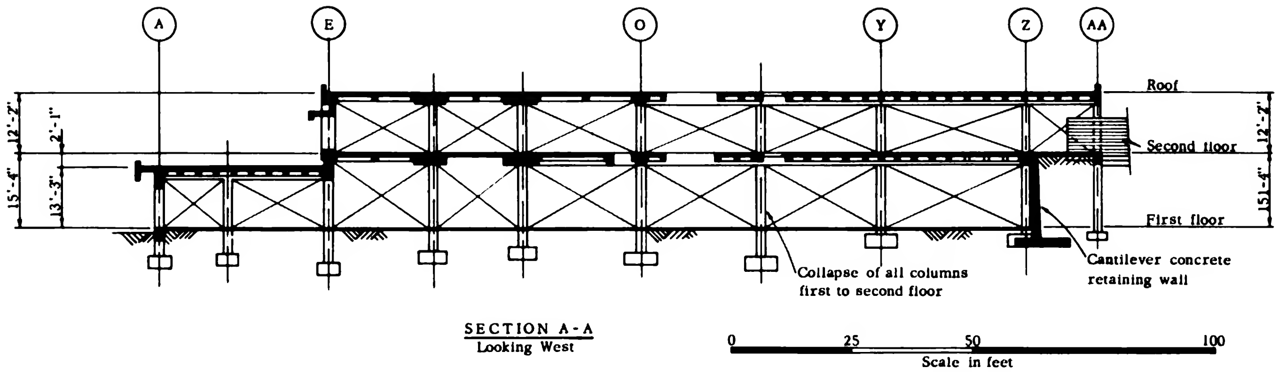

Figure 70.—Olive View Hospital, psychiatric unit. Building cross section.

Figure 71.—Olive View Hospital, psychiatric unit. Second-floor view of northeast corner, showing distress to concrete columns and beams around nonshear resisting walls. This building collapsed completely at lower story.

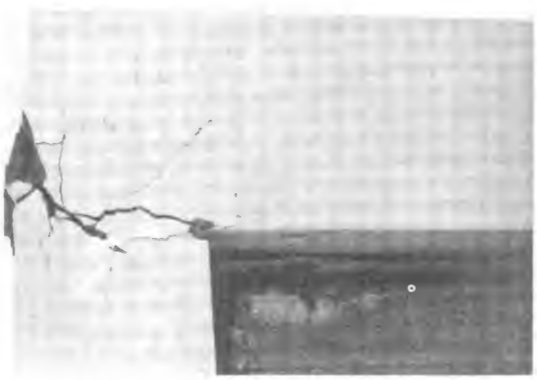

Figure 72.—Olive View Hospital, psychiatric unit. Closeup view of beam-to-column connection.

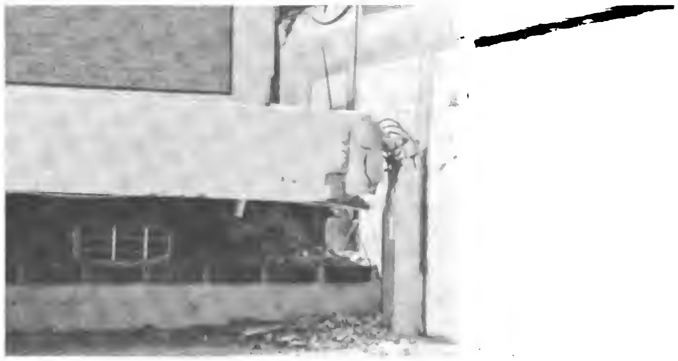

Figure 73.—Olive View Hospital, psychiatric unit. View of northeast corner where the building separated from north retaining wall. Note cracks in wall and position of beam at left relative to its original position next to retaining wall.

Figure 74.—Olive View Hospital, psychiatric unit. Interior view of north wall, looking east. Floor at right was originally at top of wall.

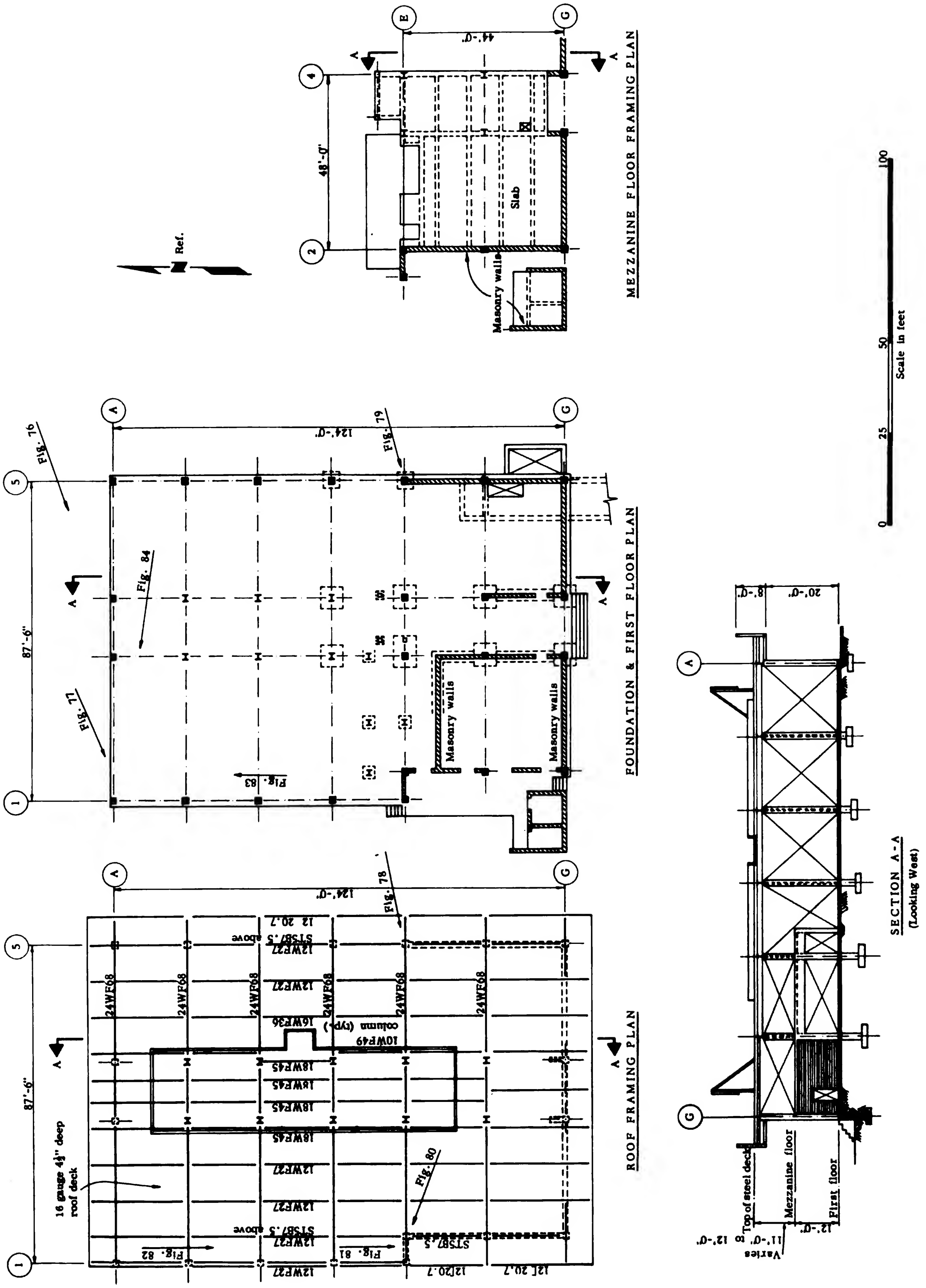

Figure 75.—Olive View Hospital, heating and refrigeration plant. Plans and building cross section.

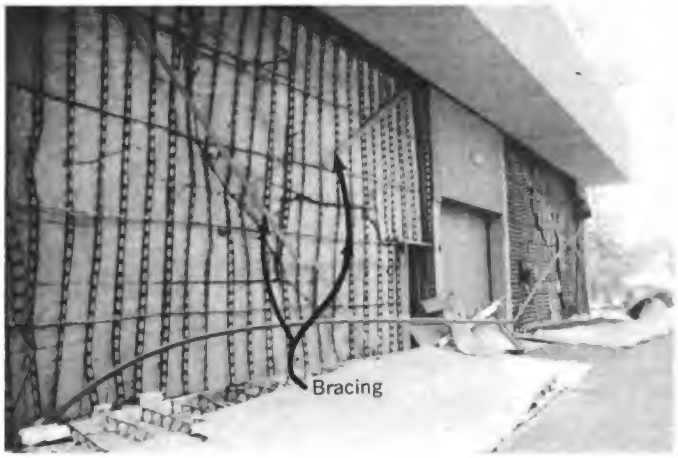

Figure 76.—Olive View Hospital, heating and refrigeration plant. View of north wall, showing failure of diagonal bracing straps and plastered stud wall on ground.

Figure 78.—Olive View Hospital, heating and refrigeration plant. Distress to upper column connection of beam and wall on east side. Note loss of ceiling below roof deck.

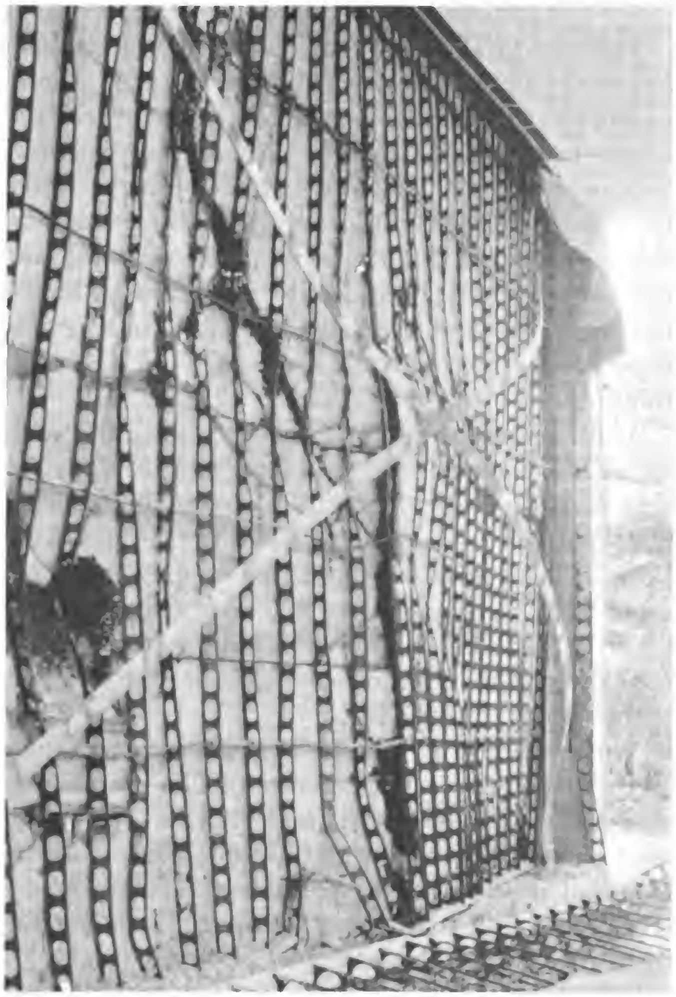

Figure 77.—Olive View Hospital, heating and refrigeration plant. Excessive deformation of diagonal bracing shown in figure 76. Vertical crack in plaster wall was caused by shifting of equipment within the building, as seen in figure 84.

Figure 79.—Olive View Hospital, heating and refrigeration plant. Distress to lower column connection of figure 78.

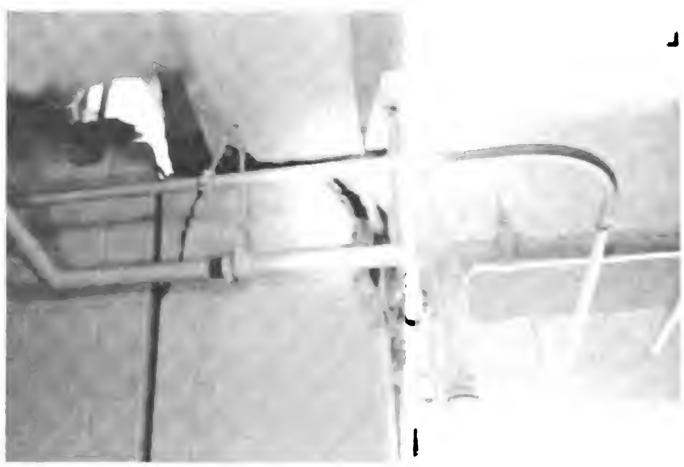

Figure 80.—Olive View Hospital, heating and refrigeration plant. Distress to upper column connection at west side.

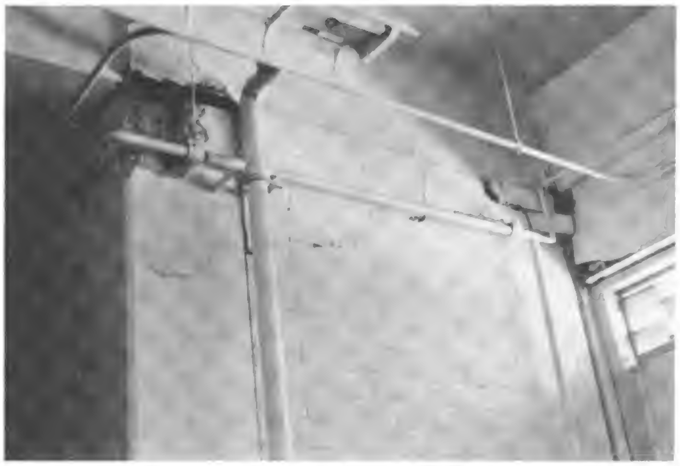

Figure 81.—Olive View Hospital, heating and refrigeration plant. Cracking in masonry wall and concrete column at west side of column shown in figure 80.

Figure 84.—Olive View Hospital, heating and refrigeration plant. Boiler final location against north wall.

Figure 82.—Olive View Hospital, heating and refrigeration plant. Distortion of west parapet.

Figure 83.—Olive View Hospital, heating and refrigeration plant. Final location of boiler on floor near west wall. Total movement was about 4 feet.

Conclusions

The structural system that resisted the earthquake forces may be described as two stories of rigid frames supporting a four-story concrete box. Whereas the upper box suffered relatively moderate structural damage, the really critical damage was in the lower two stories. From an evaluation of the observed damage, it is obvious that the columns are the most critical single element. Because the tied columns failed, the resistance of the structure to total collapse was dependent upon the resisting capacity of the spiral columns.

At the request of the Los Angeles county engineer, an ad hoc committee of structural engineers for the Structural Engineers Association of Southern California reported in a study that the ultimate lateral resistance capacity of the spiral columns was on the order of 15 percent *g* for the first story and 22 percent *g* for the second story. These resisting force levels exceed the requirements of the building code in effect at the time of construction.

The difference in ductility between spiral columns and tied columns was demonstrated in the failures that occurred in this structure. When the columns were stressed beyond the yield limits of the concrete material, it was evident that the shells outside the reinforcing cage spalled and disintegrated, leaving the cores to support the loads.

In addition, it was obvious that the tied columns did not have the shear capacity to resist the ultimate column moments. Fortunately, the spiral columns had a large reserve ductility because the concrete

within the core was confined by the close spacing of spiral reinforcing. The amount and spacing of the spiral reinforcing, selected because of empirical code requirements, provided sufficient shear resistance capacity. For example, a typical 27-inch section with a 23-inch core, reinforced with ⅝-inch spirals and spaced 2¼ inches on center, can develop the ultimate movement capacity of the sections; however, the same section reinforced with ⅜-inch ties spaced 18 inches on centers will fail in shear induced by the large moment from lateral loads.

The value, and need, for adequate ductility in all concrete members, if they are to survive large distortions, is well demonstrated by the behavior of these columns.

Stair Towers

Three of the four stair towers collapsed. The fourth (north) tower is out of vertical alignment, leaning to the north approximately 2 feet. Subsequent excavations adjacent to the north tower foundation revealed that the vertical pedestals of the foundations failed, causing the tilt in this structure.

In the north stair tower, the south wall was anchored into the basement retaining wall (fig. 44), which was pounded by the side lurch of the main structure. Although this pounding contributed to the tilt of the tower (fig. 45), the main cause was the failure of the columns under the north wall (figs. 46, 47, and 48). The bearing of the grade beams and the grade slab against the soil sustained the tower from overturning.

The other towers overturned because the supporting column and beam structure in the lower story failed, probably in shear. In the west tower the column failed, as observed in the pipe space below the ground-floor level (fig. 49). Inadequate confinement and low shear capacity are evident in this column.

In the collapsed south tower (figs. 50 and 51), there were other contributing factors. The columns were offset from the corners and interrupted the anchorage of the continuous column steel.

The walls of the towers were capable of resisting shear forces on the order of 40 percent *g*. However, the tied columns, obviously weaker elements, were subject to brittle shear failures when they could not develop the ultimate moment capacity of the members. Three towers overturned; the fourth was restrained by the soil.

Warehouse

The warehouse building is a one-story reinforced concrete structure, approximately 90 by 170 feet, and rectangular in shape. The roof consists of beam and slab framing; the exterior walls are concrete block filler walls framed between concrete columns. These block walls were designed as shear walls to resist a design load of 11 percent *g*; however, they actually were capable of sustaining three to five times this lateral load. The building, which was not damaged severely, sustained cracks in the concrete block walls.

Ambulance Canopy

The ambulance canopy is a simple, reinforced concrete two-way joist roof structure supported on concrete columns. The materials are similar to the construction in the medical treatment and care unit. The design of the rigid frames was based on a K factor of 1.33, a calculated base shear of 13.3 percent *g*, and on the assumption that the columns were pinned at the ground level. The columns were located in the center of the area between two concrete joists (figs. 52 and 53). The reinforcing steel terminated in 90° bends. Further, it should be noted that the grade-level slab extends only to the edge of the south (near) column and completely surrounds the north (far) column. The south column was free to rotate about the foundation pad; however, the lower portion of the north column remained essentially vertical. Instead of reacting as a column pinned at the base, as assumed, the north column actually behaved as a fixed base column. In the distribution of the earthquake forces to the resisting elements, the more rigid north column carried a greater portion of the lateral load. The magnitude of this force exceeded the shear capacity of the member. Consequently, the north column failed, causing the shelter to lurch toward the north and collapse. The moment capability of the columns exceeded their shear capacity as determined by current code provisions. This structure has relatively low damping, except for the small amount that was produced by shear cracking.

Exhaust Building

The exhaust building is a concrete block boxlike structure, surrounded with an independent reinforced concrete architectural canopy. The canopy-

supporting girders cantilever from the corner columns (figs. 54 and 55). The material was lightweight concrete; the design factors were $K = 1.33$; and the base shear equals 13.3 percent *g*.

The damage to the structure was primarily in the cantilever girders (figs. 56 and 57) and, particularly, in the beam-column joint. The vertical column steel lacked the proper end anchorage to resist the induced bending moment caused by lateral deformations.

There were cracks in the concrete and, in general, a considerable amount of spalling of the concrete at points of high stress and congestion of heavy reinforcing.

Assembly Building

The structural system, which resists the lateral loads, consists of two reinforced concrete columns located symmetrically near each corner of the building (figs. 58 and 59). These columns cantilever from the foundation system. Located in the top of each column is a slip-plate device that supports the concrete roof and permits it to move laterally (fig. 59,A). The concrete block enclosing walls (fig. 59) are nonbearing and structurally separated from the roof frame. The materials of construction are similar to those in the medical treatment and care unit. Lateral force design was based on a 20 percent of *g* lateral force.

The building suffered serious damage. It is evident (fig. 63) that the columns are out of plumb as much as 24 inches. Close examination reveals that the vertical column steel has yielded from severe overstress and taken a permanent set. The window wall sections, the slip joints at the tops of the columns, and many other elements (figs. 60, 61, and 62) could not resist the strain imposed by the large lateral deformation.

Walkway Canopy

The walkway canopy is a simple structure consisting of a concrete slab resting on concrete columns. It extends from the medical treatment and care unit to the psychiatric unit. Part of the structure is over the north ground floor of the medical treatment and care unit. The lateral loads are resisted by columns (fig. 63) that cantilever out of the ground and have base fixity from passive soil pressure. Materials and lateral force design criteria are the same as those in the assembly building.

The structure did not collapse fully, but considerable damage resulted from vertical ground deformation of the north ground floor of the medical treatment and care unit. This tilting caused the roof slabs to pull away from the top of the column reinforcing due to lack of anchorage (fig. 64). At the other end of the walkway at the psychiatric unit, the cantilever end of the canopy probably was broken when this building collapsed (figs. 65 and 66).

The lateral force design was based on 20 percent of gravity. Failure was induced by impacting with adjacent units.

PSYCHIATRIC UNIT

Description

The psychiatric unit is a two-story reinforced concrete building, roughly T-shaped in plan. The first floor has an area of about 40,000 square feet, while the second floor is reduced in area to about 23,000 square feet. The first story is on ground level and has retaining walls at the north end separated from the structure by 4 inches (figs. 67 through 70).

The floor framing, in general, consists of reinforced concrete pan-joist systems carried by concrete beams and columns. The first floor is a reinforced concrete slab on grade. The foundations are spread footings resting on natural soils.

The framing was designed on the basis of working stress design methods. Concrete was typically of lightweight aggregate (110 pcf) of 3,000 psi strength except for stone concrete, which was used for the foundations and the walls below the first floor. All reinforcing bars were deformed bars of 40,000 psi yield strength.

Lateral Force System

The building was designed as a moment-resisting frame ($K = 0.67$). The T-shaped floor plan resulted in numerous different rigid frames in both directions. The reinforced concrete block walls were considered as nonbearing walls and were designed not to act as lateral load-resisting elements. This lateral force system was designed using 6.7 percent of gravity based on working stress design.

Observed Damage

The building did not have adequate structural capacity to carry the induced lateral and vertical dynamic loads. All first-story columns failed, which resulted in the second floor dropping onto the first floor. The second floor also translated toward the south and east and rotated slightly counterclockwise when viewed from above. It is concluded (SEAOSC report, footnote 1, page 286) that the building collapsed by settling gradually over a period of 20 to 30 seconds as the columns failed.

The second-story columns were damaged severely, but did not collapse. The confinement of the masonry walls within the concrete frames appears to have been sufficient to require the walls to act as lateral-resisting elements (figs. 71 and 72). These conditions, and the fact that the yielding first-story columns absorbed much of the energy being transmitted into the structure, appeared to prevent the failure of the second story.

The first-story frames failed as a result of column shear failures caused by the large second-floor deflections and resulting high moments (fig. 73). Inadequate column ties to resist the shear forces is probably the major cause of the failures. The ends of the column ties were bent typically through 135°. When the concrete shattered, the ties became disengaged. The participation of the block walls at the first story as lateral-resisting elements was minor, since only a few walls were at this story and many of them had large openings at their ends. The retaining wall added no lateral resistance as the building moved away from it (fig. 74).

HEATING AND REFRIGERATION PLANT

The powerplant is a one-story building with a partial mezzanine floor and overall dimensions of 87½ by 124 feet (fig. 75). The framing is of structural steel, precast concrete exterior columns, and a roof of metal decking. Foundations are spread footings resting on natural soil materials.

Lateral loads are resisted by exterior nonbearing, reinforced, fully grouted concrete block walls on the south, east, and west ends of the building. Provisions for future expansion resulted in diagonal strap bracing in the north-end wall for lateral loads to balance the shear walls on the south end. The design base shear was 13.3 percent *g* on the basis of working stress design and a lateral load system of shear walls without vertical frames ($K = 1.33$). The weight of the cooling towers on the roof was about 50 percent of the total tributary load acting in lateral response. This building was designed and detailed to meet 1965 Los Angeles County Building Code requirements.

Damage to the building started with the diagonal straps in the north wall, stretching well into the yield range. The elongated straps permitted a large deformation to occur in the building before the straps became effective in resisting further motions (figs. 76 and 77). With the bracing ineffective, high torsional loads, in addition to direct lateral loads, were induced into the roof decking and side shear walls.

The high lateral loads and induced torsion caused strut loads at the east and west sides that, according to calculations, exceeded the design loads by 250 percent. These strut loads were carried by steel beams to anchors embedded in the top of the east and west block walls (figs. 78 through 80). The cracks in the wall resulting from the high strut loads are shown in figure 81. There was some uplift at the columns at the ends of these block walls, but this force did not appear to be excessive.

In figure 82, some buckling can be seen to have occurred at the west parapet. It also was observed that diagonal stress patterns occurred in the roof at the west side which indicates probable buckling in the steel decking under the roofing material.

Another area of damage of major proportions involved many heavy pieces of inadequately anchored equipment, which moved along the floor. In some cases, this movement produced damage to walls or piping (figs. 83 and 84).

CONCLUSIONS AND RECOMMENDATIONS

The extensive damage to structures in the Olive View Hospital complex clearly accentuates the stated, but often ignored, principle that the requirements for lateral load design as set forth in the building codes are minimum standards. They in no way obviate the necessity for the structural engineer to exercise sound engineering judgment. The code factors and coefficients were selected generally to satisfy a reasonable factor of safety in the strength of the various materials, average site conditions, regular building shape configurations, and relatively consistent lateral load-resisting systems. The unusual and

irregular structures should be analyzed with more sophisticated methods. However, although the "static load equivalent" method is used universally in the engineering profession, one of the lessons to be learned from a study of the San Fernando earthquake is the need for more realistic approaches.

For important structures such as a major hospital, it becomes clear that there should be in the first step of the design process a site evaluation to determine the magnitude of the geologic and seismic hazard. Structures should not be located directly over a fault system nor be subjected to unusual soil phenomenon; they must be designed to respond to the greatest earthquake intensity that reasonably might be expected during the planned life of the facility, without impairing the usability of the structure.

Certain specific recommendations are indicated from a study of the damaged structures:

1 All structural members should be designed for sufficient ductility to sustain the actual displacements of a maximum earthquake.

2 In order to avoid the occurrence of brittle failures, all members should be designed for a shear stress based on the ultimate moment capacity of the member.

3 The design assumptions must be consistent with the actual conditions and details of construction. Nonparticipating walls should not impinge on the action of the resisting frame element when displaced by the maximum ground motion. Grade slabs, dissimilar frames, and similar conditions should be evaluated properly as to their effect on the total rigidity of the structural system.

4 Separation joints must be sufficient to accommodate maximum displacements in order to avoid pounding of one building element against another.

5 All members of earthquake-resisting elements should be designed for reversal of stress that may be induced by maximum earthquake displacements. Flat slabs, which are a part of the lateral load system, should be detailed with proper continuous top and bottom reinforcing steel and adequate shear capacity to transfer moment capacity at columns.

6 Evidence indicated that the effectiveness of tied columns would increase materially if the spacing of the ties was close enough to intersect any possible crack formation and to provide confinement, and if the closure of the ties was anchored sufficiently into the confined concrete core.

7 The importance of proper anchorage and embedment was demonstrated by the failure of many cantilever beams and columns.

8 Tension members as a part of a lateral force bracing system should be designed to accommodate the elongation and displacement of the member and the impact imposed by the reversal of loading.

9 Equipment must be anchored to the structure.

The structural systems that are most effective in resisting the ground motion of maximum earthquakes are those which are regular, symmetrical, predictable, and subject to accurate mathematical analysis and modeling. There must be a consistent uniformity in plan, avoidance of abrupt changes in stiffness, and proper execution of design and detailing. This approach can be achieved only by full cooperation between the architect and the structural engineer.

Site Studies Olive View Hospital

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Two types of ground breakage occurred: (1) settlement of old uncompacted fills; and (2) surface cracking of pavements. However, the ground under pavements did not crack, because the pavements are more brittle than the supporting ground.

Both of these effects usually are observed in earthquakes and must be expected. What is important is that no evidences of faults, fissures, cracks, or ground displacements were found at the site except for the two minor, and predictable, cases noted above.

There is no surficial or near-surface evidence for the existence of a potentially active fault through the site. Therefore, on the basis of the geologic work accomplished in connection with this study, ground rupture from active faulting is not anticipated in the future for the Olive View Hospital site.

Exposures of the North and South Olive View faults show no evidence of recent movement. Fracture zones within each of the bedrock units apparently were not activated. There are no recent topographic scarps along fault and fracture traces. North of the Olive View Hospital area, no scarps were seen in the vicinity of the poorly exposed trace of the Hospital fault.

In the immediate area of the new hospital there is considerable evidence, particularly in paved areas, for displacement of slabs with respect to one another. However, these displacements do not involve directly the ground beneath, indicating that the pavement or concrete separated from the ground during the heaving and shaking. The strong vertical components of ground acceleration recorded during the earthquake undoubtedly added significantly to the separation of pavement from the ground.

High-angle offsets, linear depressions, bulges, and mole tracks were found west and north of the new hospital. These corresponded to the west and north edges of the collapsed and buried ground floor of the building. An east-west bulge south of the psychiatric day care center overlies a buried 8- by 8-foot steam

tunnel. Small vertical offsets west of the powerhouse and northeast of the covered parking shelter appear to result from differential settlement.

Los Angeles county survey has provided a detailed survey of Olive View Hospital that shows numerous ground elevations before and after the earthquake in an 800- by 800-foot area surrounding the hospital. Elevation changes were examined closely to determine if any significant tilting or bowing was caused by the earthquake. Existence of such tilting might indicate possible deep-seated faulting associated with the site. The review showed no significant trends in elevation changes. No discernible pattern could be established, other than an overall local uplift of approximately 1.6 feet.

All actual ground displacements at Olive View Hospital resulted from slope and ground failure. These are of minor significance at the new facilities. The largest observed is the southward creep of the planted slope east of the northern stair tower, resulting in southward tilt of light poles, tensional cracks in pavement at the sound end of the parking lot just north of the slope, and cracking of the retaining wall.

Slope failures and ridge shattering were common along the crests of steep ridges in the foothills north and northeast of Olive View. Shattered ridges and surficial slides constitute the main earthquake effects observed in the hills behind Olive View Hospital.

Summary and Conclusions for Hospitals and Medical Facilities

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The preceding building reports, 18 through 26, contain specific conclusions and recommendations. The following points are a general summary to this section.

Hospital buildings and medical facilities are a class of structure especially important to society. Not only are the patients incapacitated in many cases and unable to take, perhaps, even simple precautions to protect themselves, let alone safely endure an interruption in care, but also medical facilities are needed urgently in the hours following widespread destruction and injury due to an earthquake. At the time of disaster, these installations must be functioning, rather than being among the casualties.

Hospitals, compared with other types of buildings, are special in that they, in general, will contain an unusually large quantity of specialized, expensive, and delicate equipment and materials. Often such items, if damaged, cannot be replaced quickly or inexpensively. Indeed, hospitals themselves are neither financed easily nor quickly built in the first place. Repair or replacement required by earthquake damage has placed a severe financial strain on some if not all of the institutions.

Serious consideration should be given to providing increased levels of safety for these important and expensive facilities.

These aspects of medical facilities are important, but the evaluation of physical damage that was inflicted on the buildings themselves also is important to advance the practice of engineering. Damage to structure and equipment was extensive in most of these buildings and affords a special opportunity for study of many materials and systems.

High–Rise Buildings With Strong–Motion Instruments—Dynamic Analyses

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BACKGROUND

There were 66 high-rise buildings in the major Los Angeles area that were instrumented with strong-motion accelerographs at the time of the February 9, 1971, San Fernando earthquake. Of these instrumented buildings, 57 were within the limits of the city of Los Angeles and had been instrumented under the Los Angeles City Building Code requirement of 1965. All of these buildings within the city were designed under modern earthquake codes and constructed after July 1965. Thus, they represented the latest in engineering and construction practice.

The primary objectives for instrumenting these high-rise buildings were twofold.

First, the strong-motion accelerograph would provide the building owner, design engineer, and city safety inspector with a record of the building's response to the earthquake. By analyzing the record, the degree of potential hidden damage in the structure may be determined, thus providing a positive means for evaluating the risks to occupants of the building.

Second, the recorded motions of a building would provide valuable data for review of earthquake design procedures and evaluation of the actual safety factors in the minimum design requirements specified by the building code. The results of such reviews and evaluations hopefully would bring about improved design procedures for earthquake-resistant buildings and greater assurance of structural safety through improved minimum design requirements.

There are several secondary objectives behind strong-motion instrumentation. One is to study the influence of site conditions on the ground motions delivered to the building. Do certain sites present a greater earthquake hazard than others? What building types are particularly vulnerable under given site conditions? By instrumenting many types of buildings on various sites with different geological charac-

teristics, it is possible that the answer to these questions may be determined.

Another secondary objective is improved analytical procedures and mathematical modeling techniques. By applying the ground motions recorded at the base of a building as a forcing function to a mathematical model of the building, the engineer can recreate the motions, forces, and stresses in the building by sophisticated computer analysis techniques. The accuracy of his solution is measured by the closeness of fit between recorded and calculated acceleration response in the upper stories of the structure. The mathematical model is a primary variable affecting the accuracy of the solution. Thus, strong-motion records can provide valuable information on the credibility of mathematical models and analytical procedures.

The NOAA/EERI Subcommittee on Instrumented Buildings was formed following the San Fernando earthquake to investigate and report on the data obtained from the 66 instrumented buildings. The subcommittee formulated the following objectives or goals:

1 To conduct in-depth investigations of as many buildings as time and funding would permit.

2 To perform sophisticated dynamic analyses on mathematical models of the selected buildings, using the ground motions recorded at the base of the structures as the forcing functions.

3 To correlate analytical results from the dynamic analyses with observed and measured earthquake performance of the buildings, thus establishing credibility in the mathematical model and analytical solutions.

4 To review building performance in light of current design and construction practice.

5 To report the results of the investigation and recommend areas of further study and possible design consideration.

SELECTION OF INSTRUMENTED BUILDINGS

Because of limited time and funding the subcommittee could not conduct an in-depth investigation into each of the buildings. Instead, certain key buildings were selected for study based on the following factors:

1 The degree of structural and nonstructural earthquake damage.

2 The availability of good-quality strong-motion records from all three of the building instruments. Only 29 of the 57 instrumented buildings in Los Angeles had valid records at all three levels.

3 The availability of the records in digitized form in time for the investigation.

4 The type of structural system used in the building to resist lateral earthquake forces. An attempt was made to select structures constructed from concrete and steel and to include as many different types of lateral force systems in both of the construction materials as possible.

5 The building height and natural period of vibration. Buildings were reviewed which covered the full height range from seven stories to 42 stories and the period range from 0.3 to 3.5 seconds.

6 The regularity of structural framing was evaluated in terms of ease in mathematical modeling.

Table 1.—Pertinent information on instrumented buildings

Building report number	Building name and address	Structural characteristics			Organization performing investigation
		Number of stories	Construction materials	Lateral force system	
27	Sheraton-Universal Hotel, 3838 Lankershim Boulevard, Los Angeles.	20	Reinforced concrete..	Ductile moment-resisting frame.	John A. Blume & Associates.
28	Bank of California, 15250 Ventura Boulevard, Los Angeles.	12	.....do............	Moment-resisting frame.	Do.
29	Holiday Inn, 8244 Orion Avenue, Van Nuys.	7	.....do............	Flat slab and perimeter frame.	Do.
30	Holiday Inn, 1640 Marengo Street, Los Angeles.	7	.....do............	.....do............	Do.
31	Bunker Hill Tower, 800 West First Street, Los Angeles.	32	Structural steel......	Ductile frame (tube system).	Do.
32	KB Valley Center, 15910 Ventura Boulevard, Los Angeles.	17	.....do............	Ductile moment-resisting frame.	Conrad Associates.
33	Muir Medical Center, 7080 Hollywood Boulevard, Los Angeles.	12	Reinforced concrete..	Flat slab and perimeter frame.	Do.
34	Kajima International Building, 250 East First Street, Los Angeles.	15	Structural steel......	Ductile moment-resisting frame.	Do.

7 The location of the building, its distance from the epicenter, and relative location to other buildings under investigation were considered. Buildings were selected from as many different areas as possible.

8 The structural engineer and foundation engineer of record were considered. An attempt was made to select structures designed by different engineering firms and foundation consultants.

It also was known that certain instrumented buildings were being analyzed dynamically by other organizations and the results of these analyses would be made available to NOAA/EERI for publication. Based on these factors, the subcommittee selected the eight buildings listed in table 1. The dynamic investigations, contracted by two structural engineering firms, were performed under direct monitoring by the subcommittee.

The Sheraton-Universal Hotel did not have three good records, as the instrument at midheight malfunctioned. However, the hotel was the first building to be designed under the modern code requirements as a ductile moment-resisting concrete frame. In addition, it was the only concrete structure of this type which had records digitized in time for the investigation.

The two Holiday Inns at Orion Avenue and Marengo Street in Los Angeles were identical structures and presented a unique opportunity to study the influence of ground motion from different sites upon the same structure. Thus, the subcommittee selected identical structures in this particular case.

The Kajima International Building had been under investigation prior to the San Fernando earthquake by members of the SEAOC Seismology Subcommittee on Soil-Structure Interaction. This work was expanded to include the San Fernando earthquake effects on the building. As part of the NOAA/EERI investigation, the mathematical model of the Kajima International Building was placed on two other sites, which were located closer to the epicenter and which possessed distinctly different soil properties. The purpose of this study was to determine the influence of these parameters on the building response. A report on the Kajima International Building, *Strong-Motion Records and Simulation Analysis of KII Building in San Fernando Earthquake* by Kiyoshi Muto, also has been published by Muto Institute of Structural Mechanics.

The list of three instrumented building reports donated or purchased by NOAA/EERI for this publication and the names of the investigators are given in table 2.

The scope of the analytical investigation for the contracted buildings listed in table 1 was limited to two-dimensional linear-elastic models, which represented the horizontal component of building response in one direction at a time. A vertical earthquake analysis was performed on one typical interior column and its tributary slabs for the Holiday Inn at Orion (see Building Report 29). The scope of work was limited primarily because of time and funds. As will be noted in the building reports, further analytical work could be directed fruitfully toward the study of three-dimensional building behavior, soil-structure interaction and site response effects, structural damping on a modal basis, hysteretic effects, inelastic behavior, and ductility.

ACKNOWLEDGMENTS

The Subcommittee on Instrumented Buildings developed the scope of study, guided selection of the eight instrumented buildings investigated under con-

Table 2.—Pertinent information on instrumented buildings investigated by independent organizations and acquired by NOAA/EERI for publication

Building report number	Building name and address	Structural characteristics			Organization performing investigation
		Number of stories	Construction materials	Lateral force system	
35	Certified Life Building, 14724 Ventura Boulevard, Los Angeles.	14	Reinforced concrete..	Shear wall..........	Conrad Associates.
36	Union Bank, 445 S. Figueroa Street, Los Angeles.	42	Structural steel......	Ductile frame (tube system).	Albert C. Martin & Associates.
37	1901 Avenue of the Stars Building, Century City, Los Angeles.	19	Structural steel......	N-S-ductile frame E-W X-braced frame.	Gary Hart.

NOTE.—The Los Angeles Department of Water and Power Headquarters Building, 111 North Hope Street, Los Angeles, was analyzed by Albert C. Martin & Associates for the Structural Engineers Association of Southern California (SEAOSC). This report has been published by SEAOSC, and some of the conclusions and recommendations in that report are referred to in this section.

tract, monitored the study, and reviewed all reports before publication. In addition to these activities, seven members performed modal analyses of the buildings. The active participation and contribution from the following subcommittee members are acknowledged:

William E. Gates (cochairman), senior associate, Conrad Associates, Los Angeles, Calif.

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The strong-motion records used in the subcommittee study were digitized and supplied to the investigating organizations by the California Institute of Technology, Pasadena, under National Science Foundation funding.

Description of Analytical Procedures for Buildings Analyzed by John A. Blume & Associates

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This section (Building Reports 27, 28, 29, 30, and 31) was written under the direction of Joseph P. Nicoletti. Other Blume engineers who contributed valuable discussion time, analytical work, and field damage surveys are D. M. Teixeira, William Chaw, T. E. Bushnell, Sigmund Freeman, Ronald Gallagher, Larry Hetland, Kenneth Honda, Dilip Jhaveri, Henry Lee, Bing Mah, and Bimal Sarkar.

INTRODUCTION

The following five reports investigate the effects of the 1971 San Fernando earthquake on high-rise buildings in the Los Angeles area: Sheraton-Universal Hotel (27), Bank of California (28), Holiday Inn, Orion Avenue (29), Holiday Inn, Marengo Street (30), and Bunker Hill Tower (31).

Each of these buildings was instrumented at its roof, midheight, and ground levels with strong-motion accelerographs that recorded actual earthquake motions. In subsequent analytical studies, mathematical models of each building were subjected to these recorded earthquake ground motions, and the dynamic response of each structure was calculated so that comparisons of computed and recorded acceleration response could be made. Studies also were conducted to assess the adequacy of each building's seismic design and seismic requirements of present-day building codes.

Buildings in this investigation received only linear-elastic, two-dimensional analyses. Time and budgetary limitations prevented more detailed studies, and further analytical work could be directed fruitfully toward studies of three-dimensional behavior, soil-

structure interaction and site response, damping and hysteretic effects, and inelastic behavior and ductility (reference 1).

In the following text is a description of the principal analytical procedures and computer programs used in the dynamic analyses for Building Reports 27 through 31. The text notes specific departures from the indicated procedures where they occur. Notation symbols and references applicable to the entire report may be found at the end of this section.

In addition to the funding provided by the NOAA/EERI San Fernando Earthquake Investigation Committee, two other organizations sponsored this study. The Nevada operations office of the U.S. Atomic Energy Commission donated much of their study of the Holiday Inn on Orion Avenue. John A. Blume & Associates donated their entire study of the Bunker Hill Tower, and also partly subsidized the studies of the other four buildings.

ANALYTICAL PROCEDURES

The analytical procedures used in each of the dynamic analyses reported in Building Reports 27 through 31 closely resemble each other. These are briefly described below. Each phase of these step-by-step procedures was used for both principal directions of each building to determine both the correlation between calculated and recorded dynamic response, and to evaluate the building's structural performance under the earthquake ground motions.

Step 1: Determination of Dynamic Building Characteristics—For each building, discrete mass linear-elastic mathematical models were developed internally in computer programs to model dynamic building characteristics in each principal horizontal direction. The properties of each model were selected to closely approximate the mass and stiffness distribution of the actual structure. All structural elements that contributed to a building's lateral stiffness were considered.

Masses were calculated by determining the sum of the actual tributary dead weights at each floor, including an allowance for the estimated actual weight of furnishings, mechanical and electrical equipment, exterior walls and windows, and partitions. Member stiffness properties of moment of inertia, shear area, and cross sectional area were determined for the girder, column, or shear wall elements of each lateral force-resisting system. These properties, with story heights, bay widths, and elastic moduli, were then used as input stiffness data with the computer programs for each dynamic analysis.

The computer programs used in the analyses only allowed for one value of the modulus of elasticity to be assigned to each different lateral force-resisting frame. Consequently, the calculated moments of inertia, cross sectional areas, and shear areas of columns and girders were adjusted in frames where these members were of different elastic properties. This occurred in frames with lightweight concrete girders and regular weight concrete columns.

All column and girder joints were considered to be rigid and moment resisting. Values of the modulus of elasticity were determined from references 2 and 3. To determine effective shear modulus, Poisson's ratio for concrete was taken to be 0.20.

After the appropriate story mass and stiffness data had been compiled, a computer analysis using either the FRMSTC–4 or the FRMDYN–5 programs was performed to determine the principal dynamic characteristics, including periods of vibration, modal participation factors, and normalized mode shapes.

In each analysis, the base of each mathematical mode was considered fixed. Possible effects of soil-structure interaction, assumed to be negligible, were not included.

Step 2: Analysis of Recorded Motion—Using digitized acceleration time-history records of the actual earthquake motion, time-history plots of recorded accelerations were developed for both transverse and longitudinal directions, at both the roof and midheight levels of each building. From these plots, the fundamental period of the structure, while it responded to the earthquake, was estimated. This was done by reviewing the entire length of each record for periodicity. The periods computed in step 1 and those estimated from the actual records were then reviewed for correlation.

Step 3: Correlation of Recorded and Calculated Response—In this step, recorded earthquake accelerations at the roof and midheight levels were compared with those calculated with the mathematical models for each direction. Input motion to the models consisted of digitized records of the actual earthquake ground accelerations as recorded at the ground-floor or basement level of the structure. Either the FRMDYN–5, MATRAN, or SMIS–4 computer program was used in subjecting the mathematical models to the digitized earthquake excitation.

For most buildings several mathematical models were used. The acceleration response at both the roof and midheight levels was computed for each model. Initially, the periods calculated in step 1 were used in this step, but most models required adjustments in the initial member stiffness properties to improve period correlation. Participation factors and mode shapes assumed for the various models were taken from the work of step 1.

Correlation of computed and recorded acceleration time histories was accomplished by adjusting member stiffness properties, thereby changing the periods of vibration of the mathematical models, and by varying damping to produce calculated response that best matched the amplitude of the recorded motions. To facilitate the comparison, a plotting routine plotted both computed and recorded acceleration time histories.

Step 4: Calculation of Total Building Response—Once satisfactory correlation of response was achieved under step 3, total response, including member forces, was calculated using the FRMDYN–5 program. Total response calculations determined maximum values of story displacements, interstory drifts, story shears, overturning moments, and story accelerations.

Step 5: Determination of Structural Adequacy—In this step, either ultimate or full plastic capacities of selected members of each lateral force-resisting system were computed. Then, these were checked against the combined loading condition of maximum seismic and estimated actual vertical load effects.

For steel frames, full plastic capacities of selected members were computed on the basis of the recommendations of reference 2. Ultimate capacities of reinforced concrete members were calculated on the basis of reference 3 (without use of capacity reduction factors).

The purpose of this step was to compare nominal ultimate or full plastic member capacities with calculated combined vertical and seismic member force to determine if yield or failure conditions had been exceeded.

Step 6: Code Seismic Analysis—Finally, code seismic forces consisting of story forces, story shears, and overturning moments were determined under the requirements of the Uniform Building Code (reference 4). Then, these were compared to similar parameters calculated in the dynamic analysis. This provided a comparison of minimum code seismic forces with those estimated to have been experienced by the structure.

COMPUTER PROGRAMS

Several computer programs and plotting routines were used in the seismic study. The four most significant are described briefly. The first three were developed originally at the University of California, Berkeley, under the direction of Professor Edward L. Wilson, but later modifications and additions were made by John A. Blume & Associates. The fourth, MATRAN, was developed under the direction of Dennis D. Millerick of John A. Blume & Associates.

FRMSTC–4: Static Load Analysis of High-Rise Buildings—This static analysis program can calculate the principal dynamic characteristics of periods of vibration, mode shapes, and modal participation factors of a symmetrical high-rise building. The program uses a linear-elastic, two-dimensional analysis with rigid diaphragms assumed at each floor level.

Symmetrical nonprismatic girders, shear walls, and diagonal lateral force-resisting elements are possible. Shearing, axial, and bending deformations in columns and bending deformations in girders are considered.

Required input to the program consists of the geometry of the various lateral force-resisting frames; beam and column element stiffness properties, such as moments of inertia, elastic moduli, and cross sectional areas; and story masses. From these input data, the program develops a mathematical model of the structure internally. It then solves for the principal dynamic characteristics.

SMIS–4: Symbolic Matrix Interpretative System—SMIS–4, a general-use manipulative matrix computer program, provides a versatile means of performing matrix operations under the control of a sequence of punched cards. A program reads and executes operations as they are encountered. The program can perform a number of linear-elastic structural response analysis calculations, including calculation of eigenvectors and eigenvalues, response spectra, and time-history response. This program is useful particularly in performing complex time series analysis of earthquake time-history data.

FRMDYN–5: Dynamic Analysis of Multistory Buildings—This program can determine the dynamic response of symmetrical high-rise buildings subjected to arbitrary ground motion. The program

treats planar (two-dimensional) frames by a modal (linear-elastic) dynamic analysis, which assumes rigid diaphragms at each floor level.

Displacements, accelerations, story shears, overturning moments, and maximum member forces are computed for an arbitrary forcing function. Maximum responses at each floor level, corresponding to the individual effects of the first and higher modes, can be given as separate output. This provides a means to estimate the contribution of each mode to total response.

Symmetrical nonprismatic girders are possible, and the shearing, axial, and bending deformations of columns and bending deformations in girders are considered. The program also considers column widths and story heights. Forces are computed at column and girder faces. Some of the analytical techniques used in the program are described in reference 5.

MATRAN: Matrix Analysis System—MATRAN is an array manipulation and structural analysis system designed for performing dynamic analyses of unique structures. The system includes assorted instructions of data input and output, array manipulation, matrix algebra, structural analysis, and time series analysis. These instructions may be performed individually or collectively as macro-instructions for standard or repetitive procedures. A program run consists of two passes. During the first pass, the input instruction is checked for syntax and compatibility. If no errors are detected, the instructions are executed in the order read during the second pass.

NOTATION

A_g = Gross cross sectional area for girders and columns, or gross cross sectional area of 50 percent of the tributary slab area between bays for slabs
A_{web} = Area of web
B = Beam width
C = Uniform Building Code (UBC) numerical coefficient, dependent upon fundamental period of vibration of structure
D = Overall beam depth
E = Modulus of elasticity
f'_c = Specified minimum compressive strength of concrete at 28 days
f_y = Minimum specified yield strength of structural or reinforcing steel
g = Acceleration of gravity
I_o = Moment of inertia of gross section for girders and columns, or moment of inertia of 50 percent of gross tributary slab area between bays for slabs
J = UBC numerical coefficient for base overturning moment
K = UBC horizontal force factor, dependent on type and configuration of lateral force-resisting system
M = Bending moment
M_p = Plastic moment (Z_pf_y) for structural steel shapes
M_{pc} = Plastic moment modified to include the effect of axial compression (see reference 2)
M_x = Bending moment about strong or x-axis
M_y = Bending moment about weak or y-axis
M_u = Ultimate moment capacity (see reference 3)
M_{ux} = Ultimate moment capacity about strong axis under combined axial load and bending
M_{uy} = Ultimate moment capacity about weak axis under combined axial load and bending
P = Axial load
P_{uo} = Ultimate axial load capacity without bending moment (see reference 3)
P_y = Plastic axial load (A_gf_y) of steel members
S_a = Spectral acceleration
T = Fundamental period of vibration of structure
T_i = Period of vibration of ith mode of structure
V = Total seismic shear at the base
W = Total weight of building
Z = UBC regional seismic risk factor
Z_p = Plastic section modulus
λ = Percent of critical viscous damping
ϕ = Capacity reduction factor

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Sheraton-Universal Hotel (27)

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JOHN A. BLUME & ASSOCIATES, ENGINEERS

San Francisco, Calif.

DESCRIPTION OF BUILDING

Located in the Universal City area of Los Angeles, the Sheraton-Universal Hotel stands about 19 miles south of the epicenter of the San Fernando earthquake. This area lies in the southeast end of the San Fernando Valley near the eastern terminus of the Santa Monica Mountains.

The building, shown in figures 1 and 2, is a 20-story structure that serves as a hotel and convention center. It was completed in 1968 at a cost of $7.5 million. Plan dimensions of the central tower portion, which runs from the fourth floor to the roof, are typically 183 feet 6 inches long by 57 feet 10 inches wide. At the third-floor level, the width becomes 96 feet 4 inches and the length extends to 198 feet 7 inches. A lobby floor, ground floor, and basement of similar plan descend below the third-floor level. The central tower portion, the subject of this report, is separated seismically from the rest of the structure by expansion joints.

Reinforced concrete spread footings comprise the foundations for the structure. The underlying soil

Figure 1.—Sheraton-Universal Hotel. West elevation. John A. Blume & Associates photograph.

Figure 2.—Sheraton-Universal Hotel. South elevation. John A. Blume & Associates photograph.

deposits consist primarily of bedded sandstones with deposits of shale and clay. Design soil bearing capacity for dead plus live load was 6 ksf (kips per square foot) (reference 6). A geologic study of the bedding revealed no adverse bedding planes, but a wide fault zone crosses the site and occupies the area immediately beneath the foundations of the central tower.

Two typical soil boring logs (reference 7) are given in figures 3 and 4. These indicate the bedding conditions at the site. Boring No. 1 was drilled on the north side of the larger fault zone shown in figure 5, while boring No. 2 was drilled into the middle of the larger fault zone.

A reinforced concrete moment-resisting frame resists lateral forces in each direction, except at the basement level where 12-inch-thick reinforced concrete shear walls are employed. The building has been designed as a ductile moment-resisting frame structure, meeting the requirements of Division 26 (concrete) of the Los Angeles City Building Code as amended November 7, 1966. Seismic provisions of this code approximate the 1970 UBC (reference 4).

Typical framing consists of columns spaced at about 19 feet on center in the transverse direction and at 13 feet on center in the longitudinal direction, with interconnecting floor girders in each direction. The floor system consists of two-way reinforced concrete slabs spanning between girders. Slabs are typically 4½ inches thick in guest rooms and 6 inches thick in corridors. At the lobby and ground-floor levels, the slab thickness is 5 inches. Exterior columns on the north and south sides are not prismatic, but taper from a 20- by 18-inch section at the top and bottom of each story to a 20- by 15-inch section at midstory height. Foundation, floor, and roof plans are shown in figures 5, 6, and 7. A typical

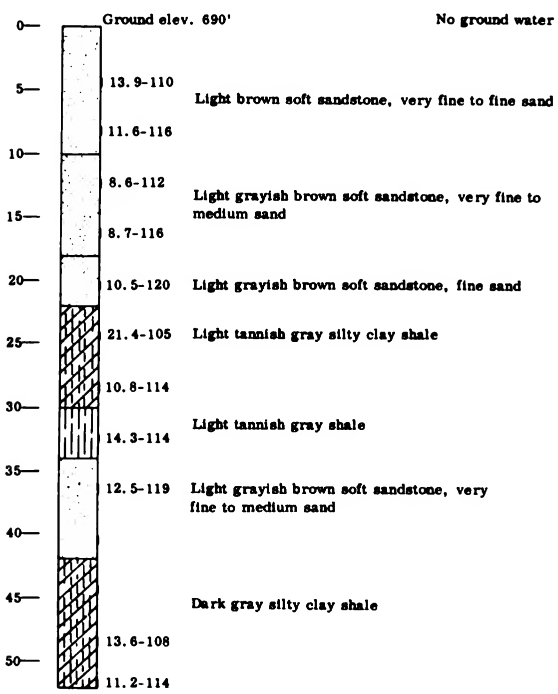

Figure 3.—Sheraton-Universal Hotel. Boring log of test hole No. 1.

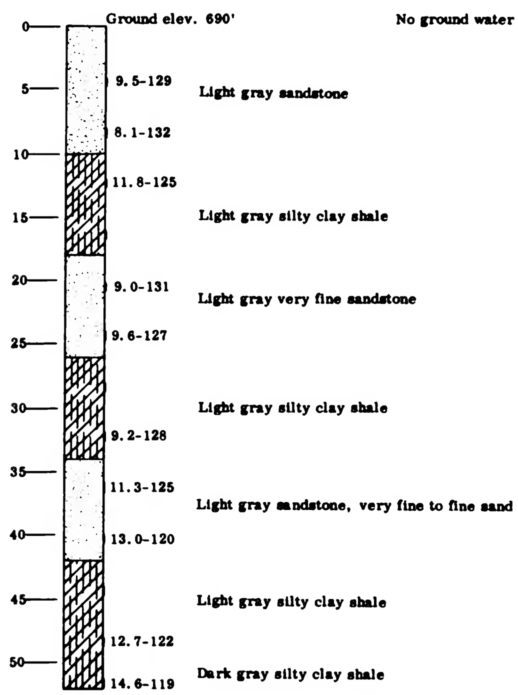

Figure 4.—Sheraton-Universal Hotel. Boring log of test hole No. 2.

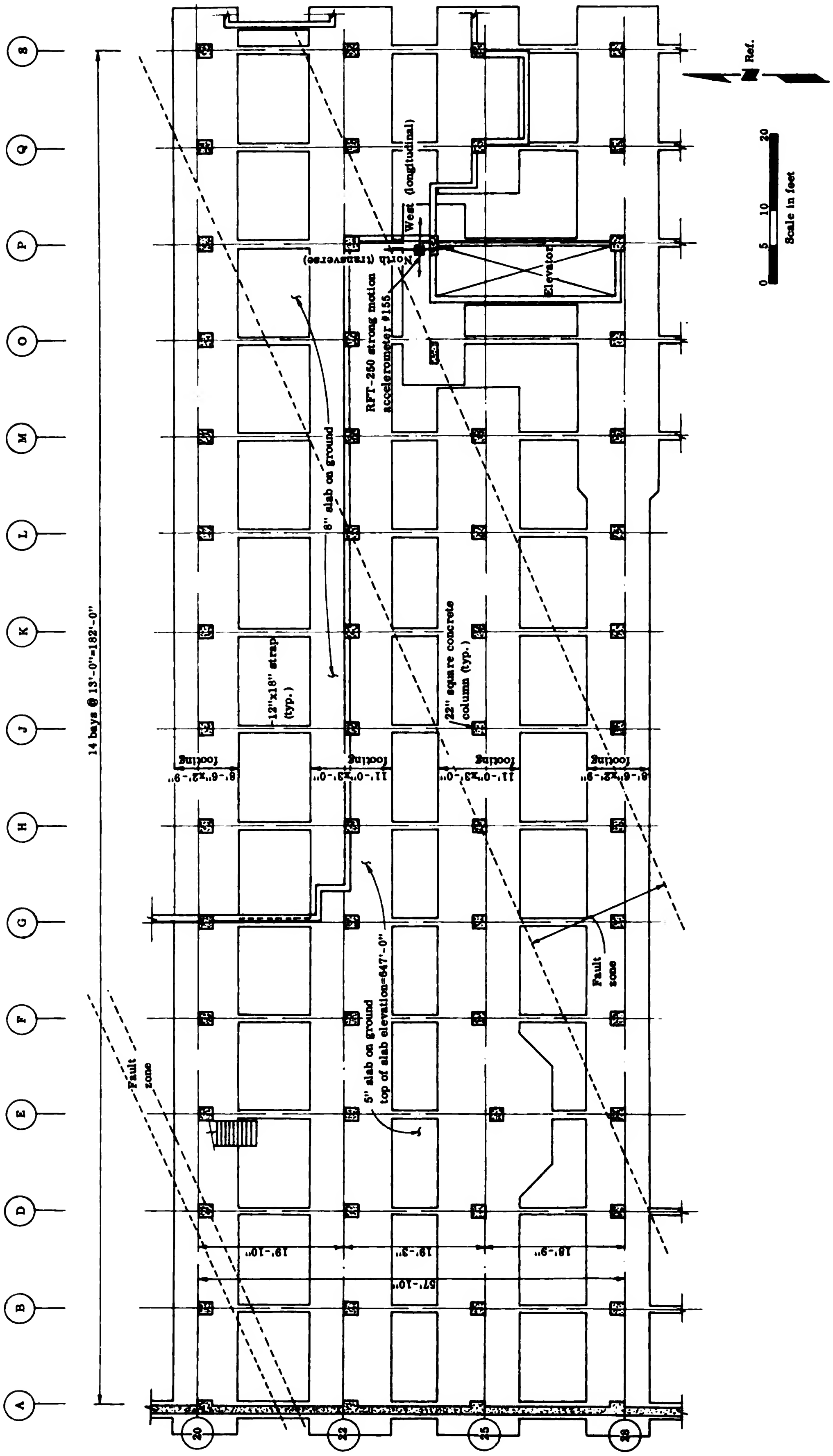

Figure 5.—Sheraton-Universal Hotel. Partial foundation plan.

elevation and key structural details are shown in figure 8.

The structure was constructed principally with lightweight concrete. Lightweight aggregates were used exclusively above the ground-floor level. Properties of the various materials used in the construction are given in table 1.

Typical interior partitions consist of gypsum wallboard on metal studs, or gypsum coreboard. Some plaster partitions are located at the ground-floor level. Walls in the longitudinal (east-west) direction were secured to the structure on all edges. In the transverse (north-south) direction, walls were separated seismically from the building frame with a ⅜-inch gap by means of a neoprene filler strip (fig. 9).

Exterior end walls in the east and west facades consist of 4-inch-thick precast concrete panels connected to the spandrel beams by strap anchors and surrounded by a ⅜-inch gap. Slotted bolt holes at the top connections, as shown in figure 8, allow lateral movement of the frames. In the longitudinal direction, the entire facade consists of glass curtain walls located between columns.

The designer followed standard inspection procedures during construction. The structural engineer provided a full-time licensed inspector to inspect construction and to interpret design drawings.

EARTHQUAKE DAMAGE

Although subjected to a 0.18*g* peak ground acceleration, the $7.5 million hotel suffered only slight architectural damage, totaling $2,100. Neither field observations nor this seismic study indicated structural damage. Figure 10 may be used to locate points of damage on a key plan of the entire building complex.

The following five points briefly summarize the known damage:

Point A. The seismic joint cover at the low roof suffered a ¾-inch permanent displacement in the east-west direction. At the lobby floor, an aluminum seismic joint cover buckled. A water seal was broken at the roof coping at both sides of the seismic joint.

Point B. The corner column at the third floor suffered minor concrete spalling, approximately 8 inches long. No reinforcing steel was exposed.

Point C. Evidence of seismic joint movement was apparent. In addition, horizontal cracking appeared at the ground floorline in this stairwell (on three sides) and at the underside of the beam in the lobby floor level above the stairwell.

Point D. At the low roof coping there was indication of slight joint movement.

Point E. A mosaic tile mural mounted on columns adjacent to the seismic joint at the ground floor was damaged, apparently by impact of the adjacent wing. About 75 tiles were knocked loose at the west end of the mural.

There also was cracking of the plaster walls on the lower level and the gypsum wallboard partitions throughout the building in the east-west direction, with a higher concentration noted on the ground floor at the west end (plaster walls). Cracking in partitions also occurred in a band that started at the 14th floor at the east end and extended to the 10th

Table 1.—Properties of construction materials

Material—Location in structure	Aggregates	Unit weight	Minimum specified compressive strength (f'_c)	Modulus of elasticity (E)
Concrete:		*pcf*[1]	*psi*[2]	*psi*[2]
All concrete, basement to ground floor	Regular weight (ASTM C-33)	150	3,000	3.3×10^6
Columns, ground floor to 10th floor	Lightweight (ASTM C-330)	110	4,000	2.4×10^6
Columns, 10th floor to roof	do	110	3,000	2.1×10^6
Beams and slabs, ground floor to roof	do	110	3,000	2.1×10^6
Material—Location in structure	**Grade**		**Minimum specified yield strength (f_y)**	**Modulus of elasticity (E)**
Reinforcing steel:			*ksi*[3]	*psi*[2]
All reinforcement, except columns	Intermediate-grade deformed billet bars (ASTM A-15 and A-305).		40	29×10^6
Columns, foundation to 10th floor	Deformed billet bars (ASTM A-432)		60	29×10^6
Columns, 10th floor to roof	Intermediate-grade deformed billet bars (ASTM A-15)		40	29×10^6

[1] Pounds per cubic foot. [2] Pounds per square inch. [3] Kips per square inch.

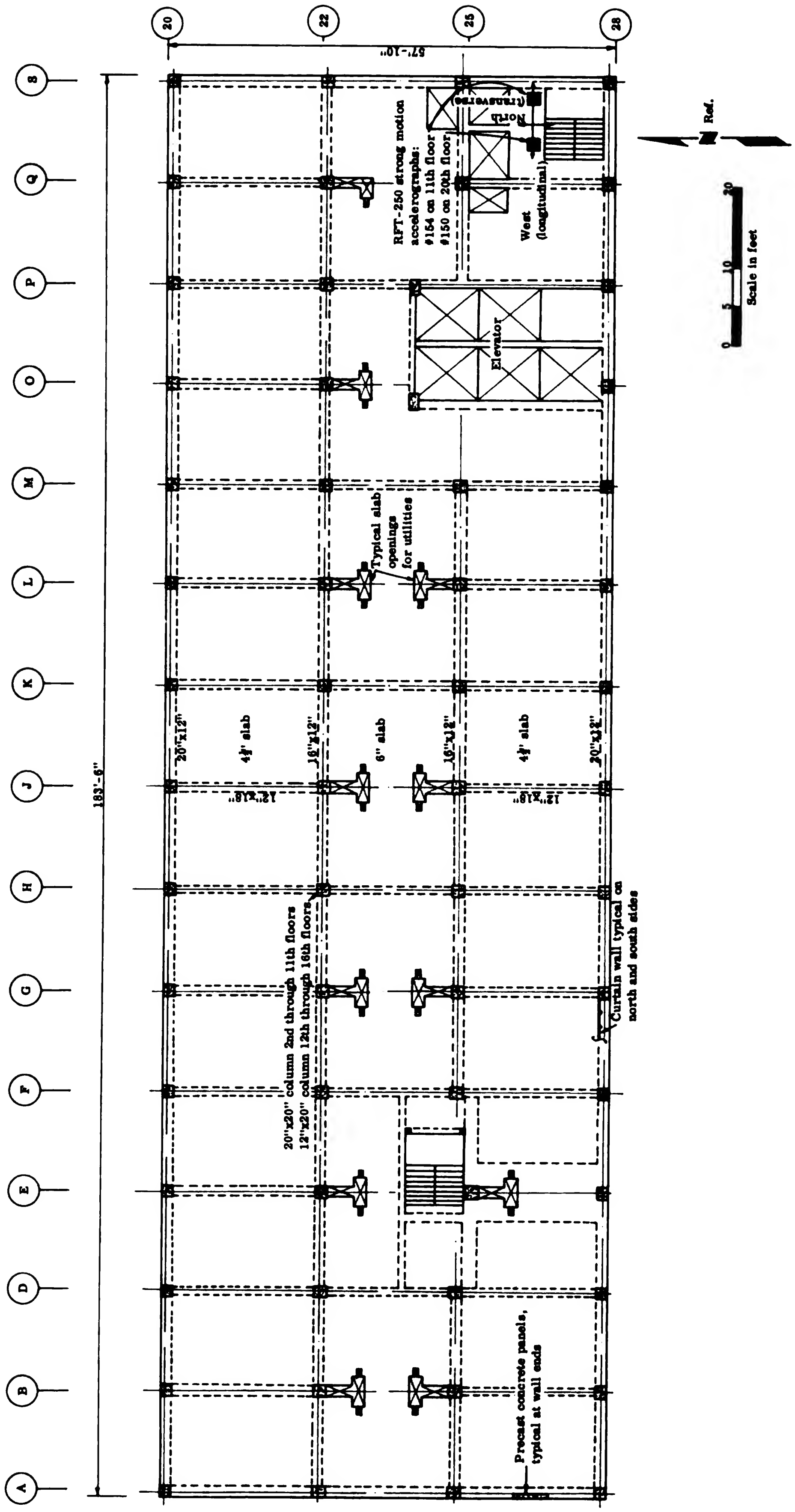

Figure 6.—Sheraton-Universal Hotel. Typical floor framing plan.

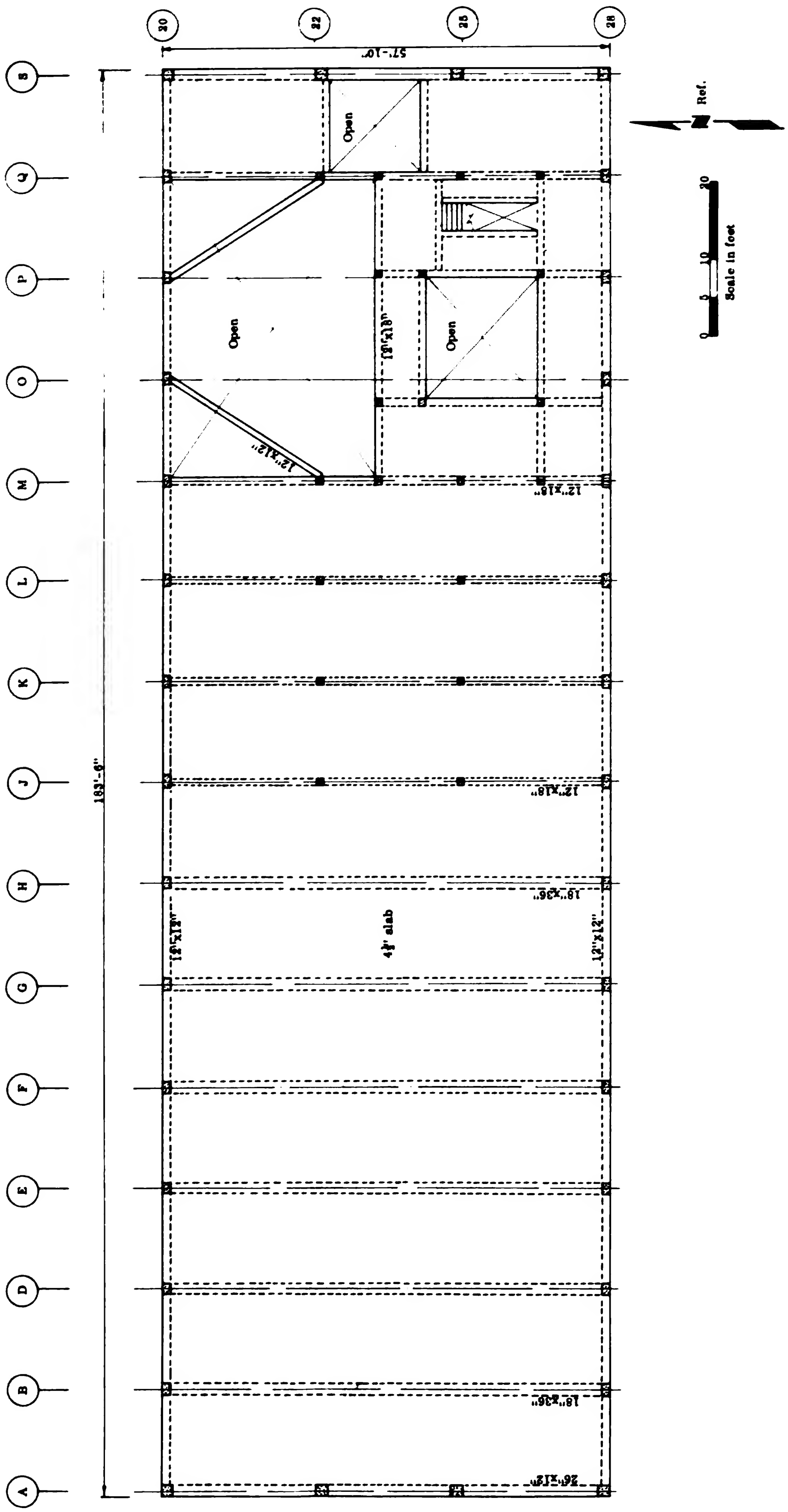

Figure 7.—Sheraton-Universal Hotel. Roof framing plan.

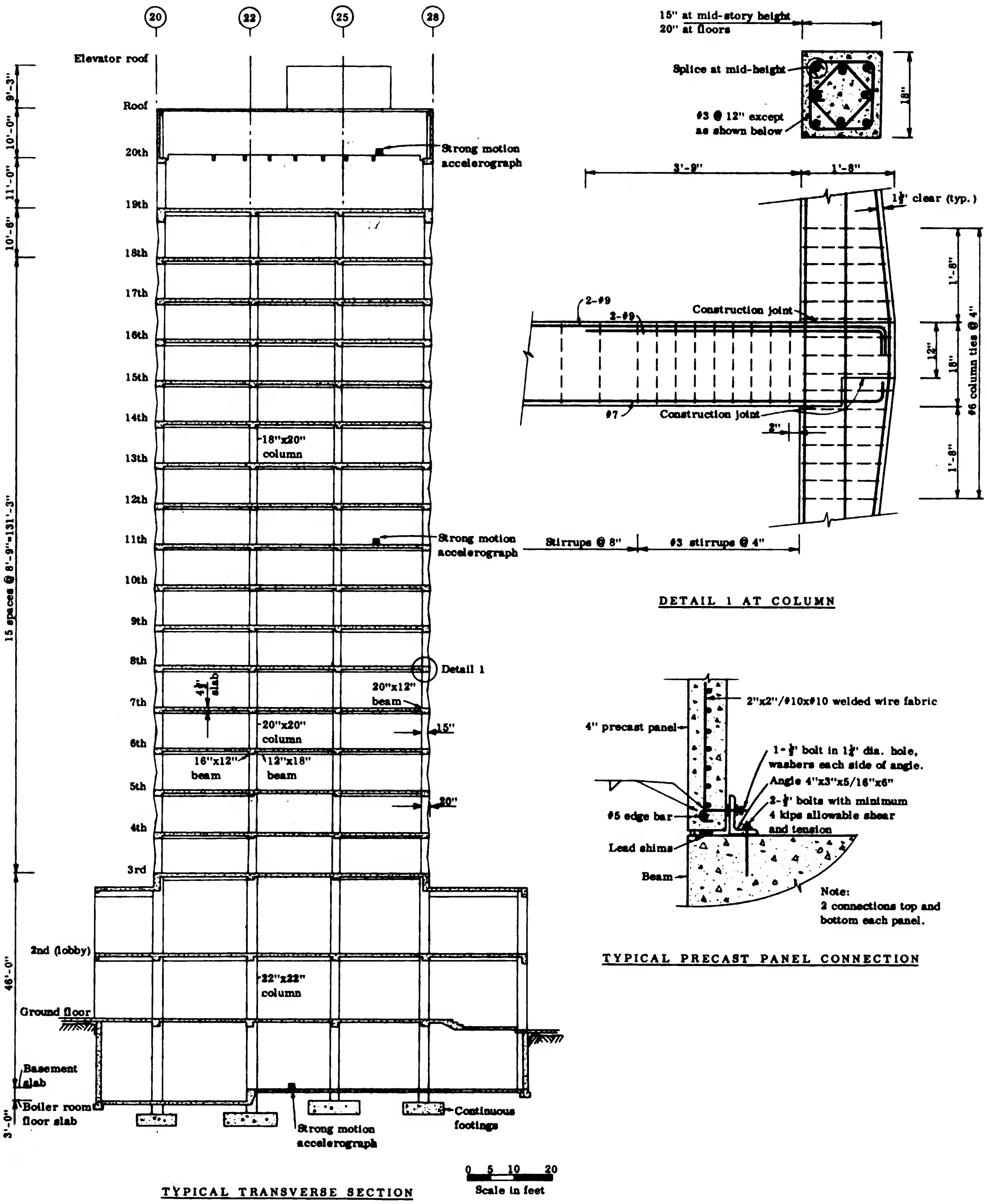

Figure 8.—Sheraton-Universal Hotel. Typical transverse section and structural details.

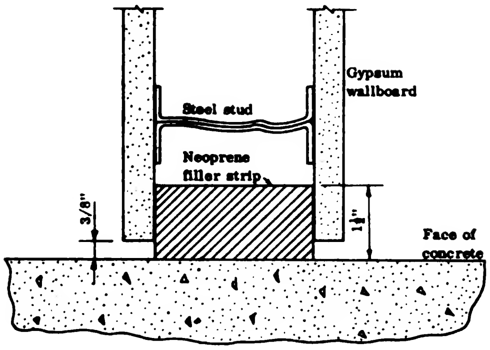

Figure 9.—Sheraton-Universal Hotel. Detail of north-south partition control joint at vertical edges. Detail at top edge is similar.

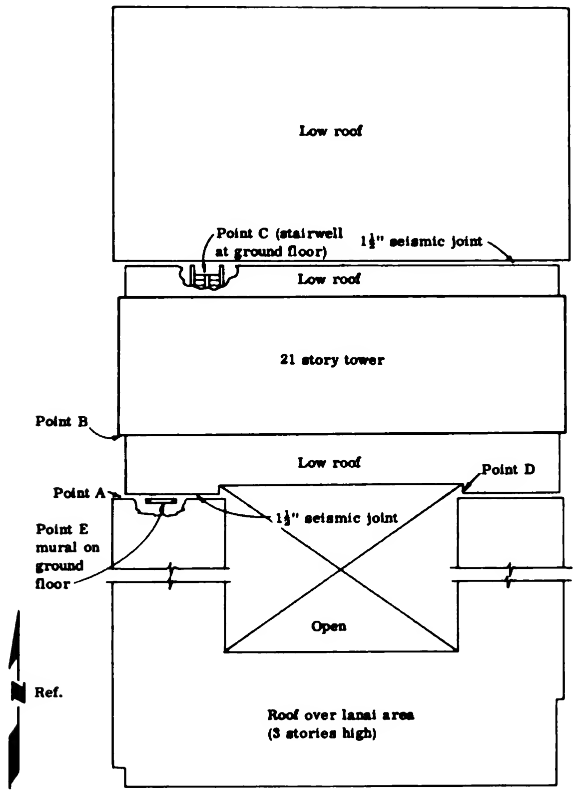

Figure 10.—Sheraton-Universal Hotel. Key plan of building showing location of damage.

floor at the west end. This band of cracking consisted primarily of diagonal cracks from the upper corners of the doors to the ceilings in the east-west filler walls on column lines 22 and 25. These walls, 2-inch laminated gypsum wallboard partitions, apparently were secured on all four edges. The major wall cracking was concentrated between the ninth and 11th floors.

RECORDED EARTHQUAKE RESPONSE

Three Earth Sciences RFT–250 strong-motion accelerographs recorded the San Fernando earthquake motions. The location of each, at the 20th floor, 11th floor, and basement levels, may be found on the floor plans in figures 5 and 6. Accelerations were recorded for both principal horizontal axes and for the vertical axis. Approximately the first 28 seconds of motion was recorded at the roof and basement levels, but due to a malfunction in starting, all motion records for the 11th floor are incomplete. Peak recorded ground and floor acceleration values are summarized in table 2. Figures 11 and 12 provide plots of the recorded accelerations.

Table 2.—Peak recorded accelerations

Station	Transverse (north/south) component	Longitudinal (east/west) component	Vertical component
	g	g	g
20th floor.........	0.120	0.195	0.260
11th floor.........	[1]	[1]	[1]
Basement.........	0.175	0.165	0.087

[1] Not recorded because of instrument malfunction.

From the recorded basement-level ground motion records, response spectra were determined for 2 and 10 percent of critical damping using the SMIS–4 program. Figures 13, 14, and 15 show these for the transverse, longitudinal, and vertical components of motion. The spectra have been drawn on four-way log paper to aid the reading of either the pseudo-absolute acceleration, pseudo-relative velocity, or relative displacement values.

MATHEMATICAL MODELING

In a manner similar to the general analytical procedures discussed previously, two mathematical models, one for each principal direction, were developed to model the physical characteristics of the lower portion of the hotel in its response to the earthquake ground motion. For each model, member stiffness properties were assumed to be those indicated in table 3. Values of the gross moment of iner-

Table 3.—Member stiffness properties

Member	Moment of inertia	Shear area	Cross sectional area
Girder.......................	I_o	....	A_g
Column.......................	I_o	5/6 A_g	A_g
Shear wall	I_o	5/6 A_g	....

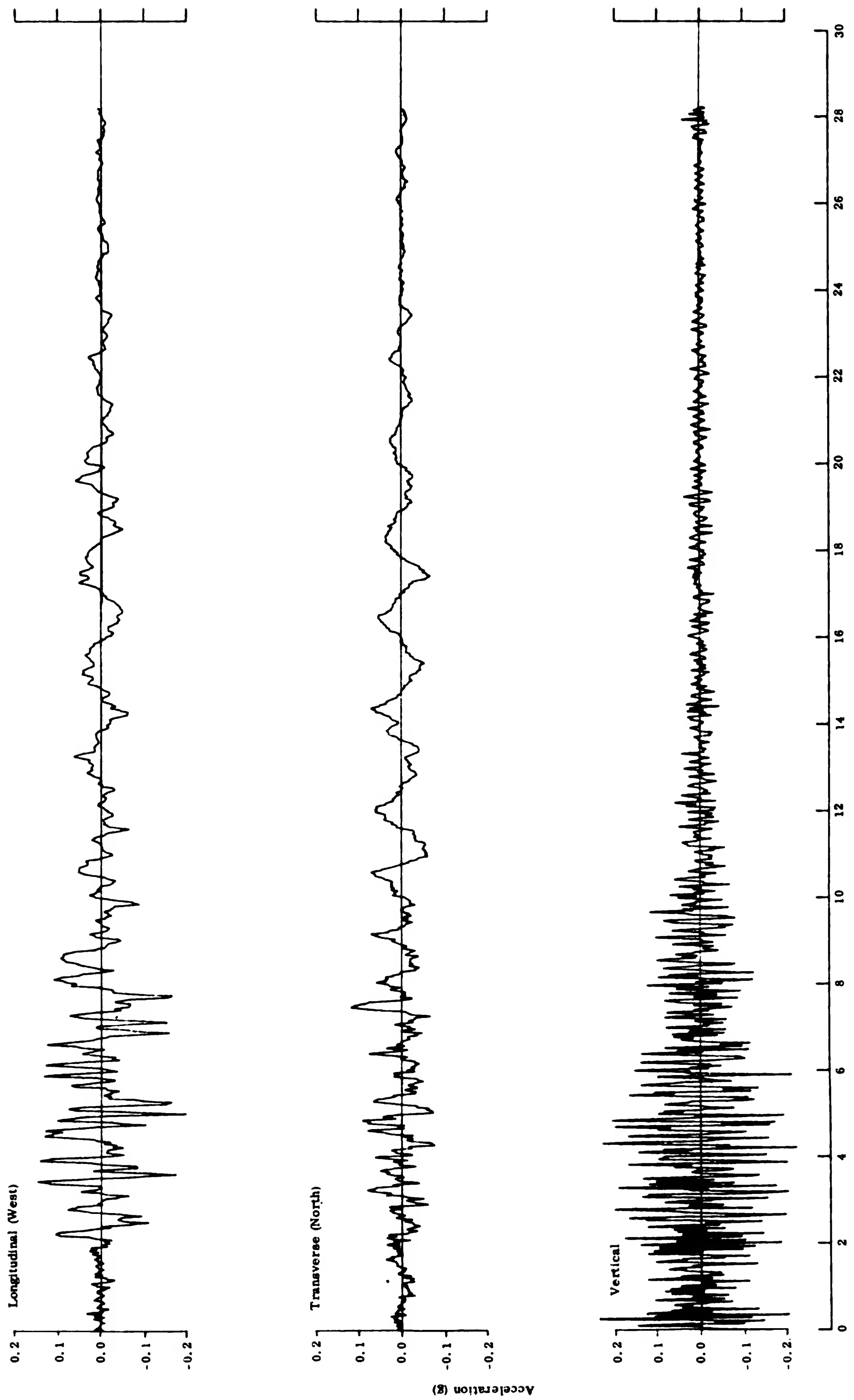

Figure 11.—Sheraton-Universal Hotel. Recorded acceleration at 20th floor.

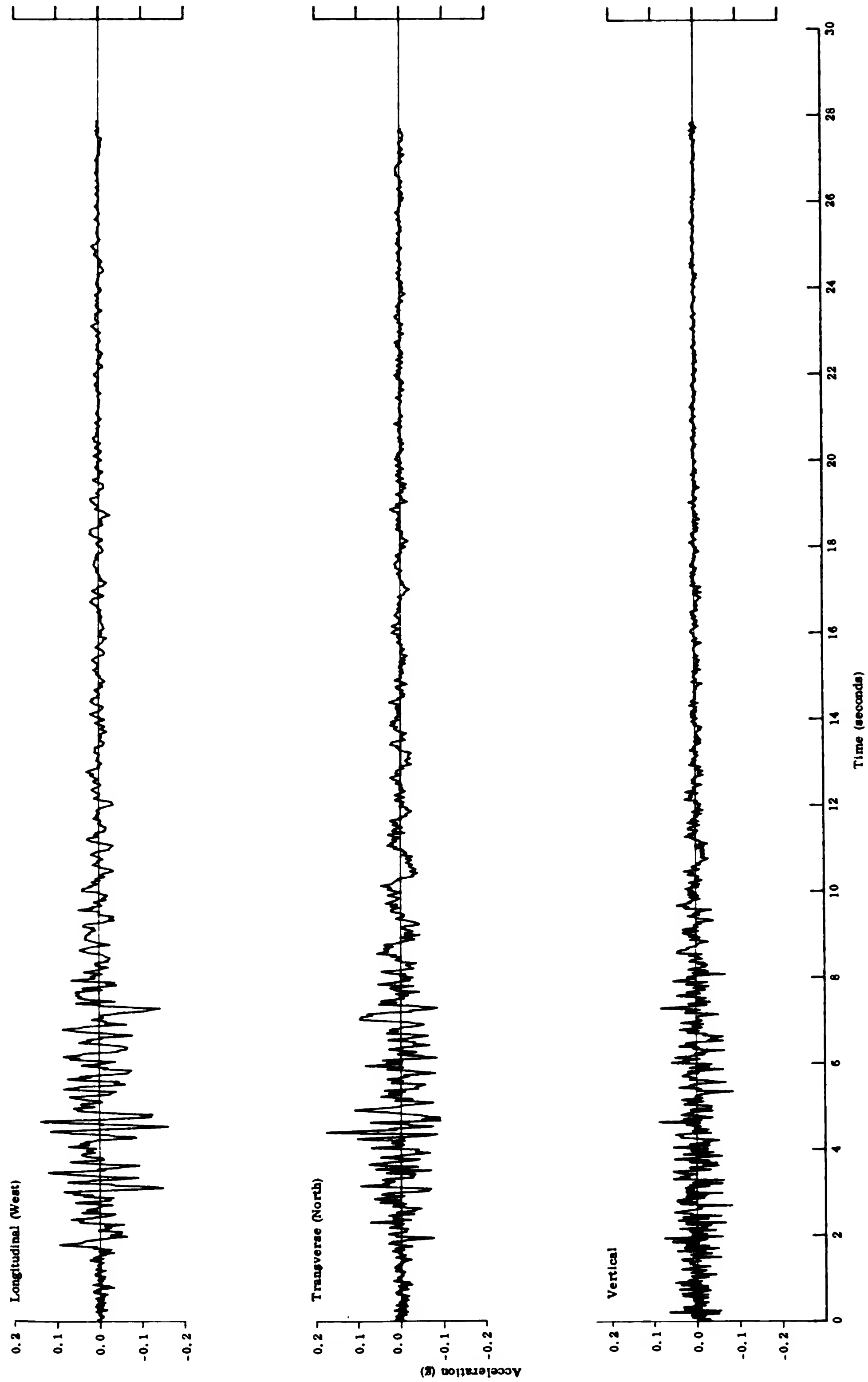

Figure 12.—Sheraton-Universal Hotel. Recorded acceleration at basement.

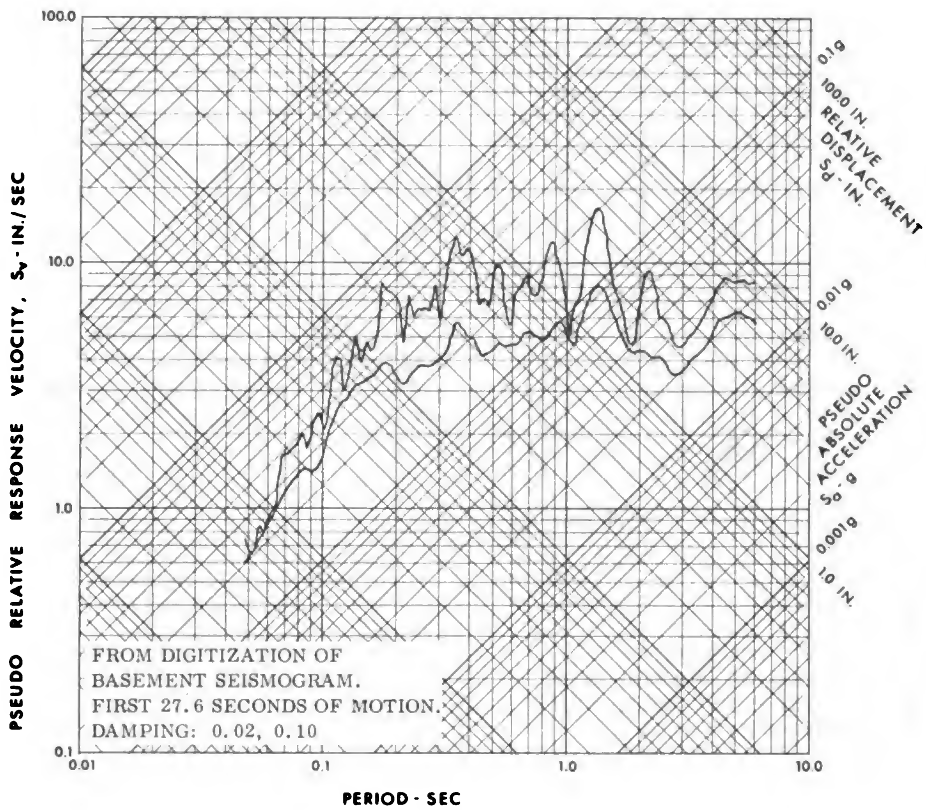

Figure 13.—Sheraton-Universal Hotel. Transverse response spectra.

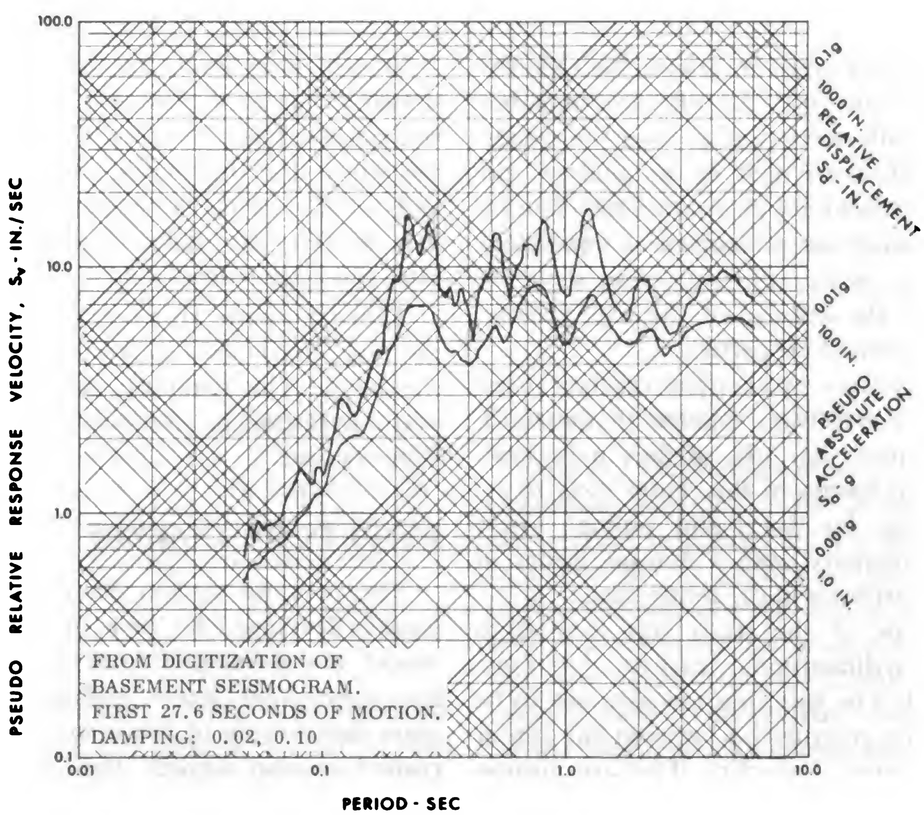

Figure 14.—Sheraton-Universal Hotel. Longitudinal response spectra.

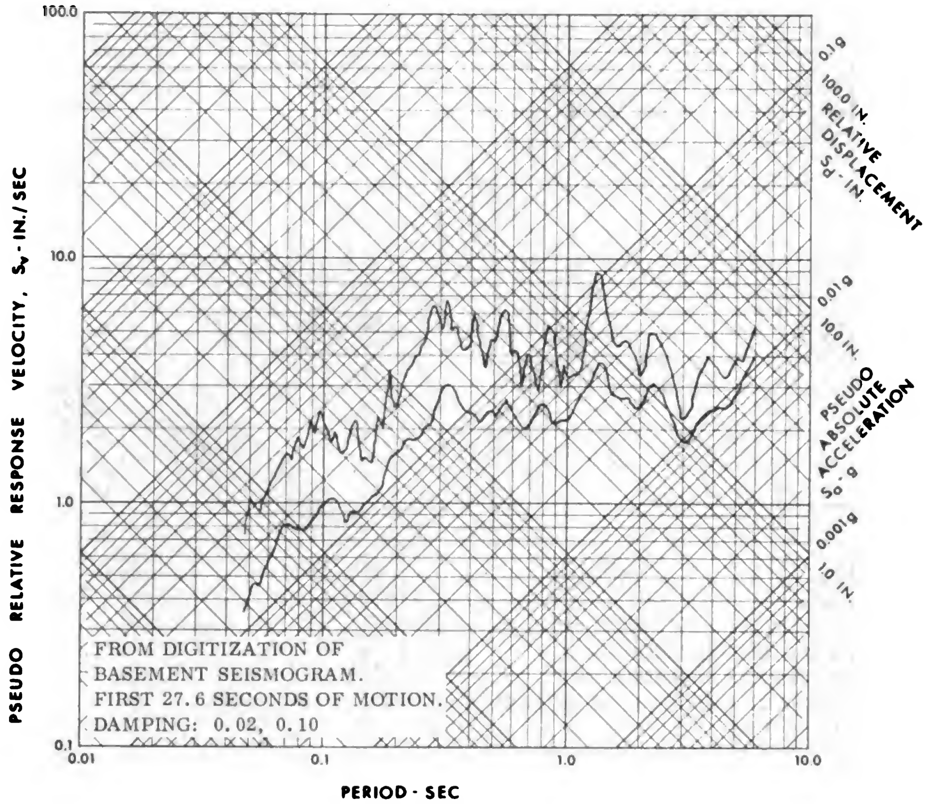

Figure 15.—Sheraton-Universal Hotel. Vertical response spectra.

tia, I_0, of each member were reviewed. Neither the effects of reinforcement nor the effective member lengths, which actually were shorter than the centerline to centerline distances used in the analyses, necessitated any adjustment for this structure. The tapered exterior columns were modeled as equivalent prismatic members, with adjusted cross sectional properties taken as the average of the top, middle, and bottom cross sectional properties.

Figures 16 and 17 show geometrical configurations of typical lateral force-resisting frames. In the transverse direction frames (fig. 16), girders have been omitted at alternate frames in the center span to accommodate openings for ducts and piping. These frames then were modeled with a dummy girder of negligible bending stiffness in the center span.

Because the scope of the study was limited to planar analyses, two-dimensional mathematical models were developed. The building was assumed to be symmetrical with no eccentricity between the center of mass and the center of rigidity. This assumption reasonably approximates the actual physical situation existing in the tower portion of the hotel.

At each story level, the concrete floor slab was assumed to act as a rigid horizontal diaphragm. This was regarded as a reasonable assumption because of the width and depth dimensions of the diaphragm, and because of the absence of any significant openings in the floor slabs. The largest width-to-depth ratio was approximately 3.2.

Table 4 briefly summarizes the characteristics of the two models used to study the seismic response of the hotel. The damping values indicated gave the best correlation of computed and recorded acceleration response.

RESULTS OF ANALYSIS

The previous section described the mathematical models developed for each principal direction. Each model was developed from a careful assessment of the anticipated actual stiffness characteristics and mass distribution of the structure. Review of the recorded motion records (figs. 11 and 12) indicated that the building, during approximately the first 6 seconds of the earthquake, responded with a higher

Table 4.—Mathematical models used in the analysis

Building direction	Fundamental period	Purpose	Earthquake time interval	Number of modes	Applied viscous damping
	Seconds				*Percent*
Transverse.............	2.22	Mode shapes, periods, and dynamic analysis.....	(1)	3	10
Longitudinal...........	2.15	.....do....................................	(1)	3	10

[1] The earthquake time interval used in the acceleration time-history correlation was 0 to 27.6 seconds. For computation of maximum response parameters (forces, overturning moments, etc.), a 0- to 22.2-second time interval was used.

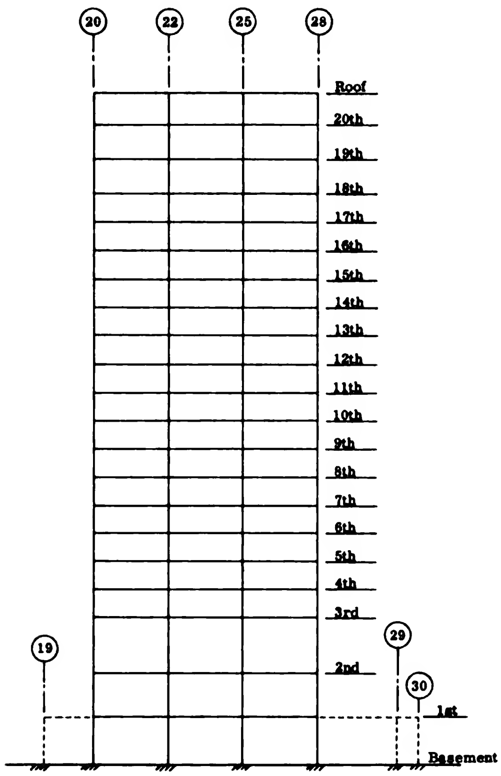

Figure 16.—Sheraton-Universal Hotel. Typical north-south frame.

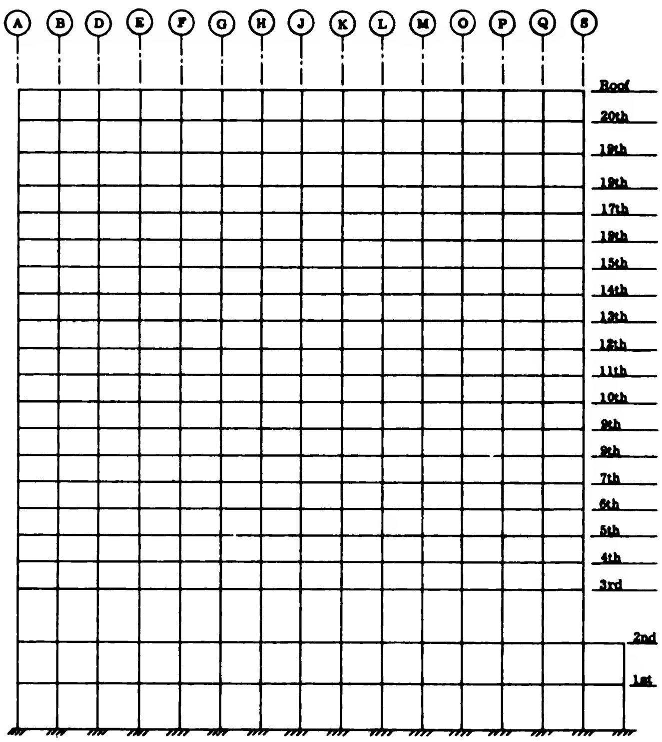

Figure 17.—Sheraton-Universal Hotel. Typical east-west frame.

frequency than that indicated by the fundamental periods of the mathematical models. For the latter part of the earthquake, the structure responded in a predictable manner in both principal directions. Reasonable correlation of computed and recorded response was achieved.

Mode Shapes and Periods of Vibration

Mode shapes and periods of vibration were calculated for the first three translational modes in both the transverse and longitudinal directions. These are indicated in figure 18.

Computed Floor Accelerations

Figures 11 and 12 show plots of the earthquake accelerations recorded in the transverse, longitudinal, and vertical directions by strong-motion instruments located at the 20th floor and basement levels. As part of the dynamic analysis, and as a check of the validity of the mathematical models, correlation studies of recorded versus computed floor accelerations were made. Figure 19 summarizes results of these studies for the roof level. No comparison could be made for the 11th floor because the first, and most important, part of the motion for this level was not recorded.

Table 5.—Maximum accelerations and displacements at 20th and 11th floors

Direction of response	Maximum parameter	20th floor	11th floor
Transverse	Computed acceleration (g)	0.118	0.047
	Recorded acceleration (g)	.120	(1)
	Computed displacement (in.)	2.4	1.4
	Computed story drift (in.)	.14	.14
Longitudinal	Computed acceleration (g)	0.155	0.082
	Recorded acceleration (g)	.195	(1)
	Computed displacement (in.)	2.6	2.0
	Computed story drift (in.)	.14	.19

[1] Not recorded because of instrument malfunction.

Table 5 provides a comparison of recorded and computed peak roof accelerations. It also indicates maximum computed displacements and interstory drifts.

Maximum Building Displacements

Envelopes of maximum total displacement are shown in figures 20a and 21a. These represent the peak displacement of each story with respect to the basement. Not all of the maximums occurred simultaneously. In the transverse direction, maximums occurred approximately 11 seconds after the start of motion, while in the longitudinal direction, maximums occurred between 7 and 8 seconds after the start of motion.

Maximum Story Forces

Figures 20b and 21b plot the maximum horizontal story forces from the dynamic analysis and corresponding code values. For each level, story forces were determined as the product of the story mass times the maximum absolute story acceleration.

Maximum Story Shears

Maximum story shears were calculated as part of the dynamic analysis. Figures 20c and 21c indicate these as smooth curves with corresponding code values. Code design shears have been computed for the requirements of both the 1967 UBC, which was used in the original design, and for the 1970 UBC, which would govern if the structure were to be built in 1971. Both codes indicated identical story shear requirements. The UBC story shears for both directions are based on the numerical coefficients and periods shown in table 6.

The dynamic analysis determined peak values for the shears. These were calculated to have occurred at each floor level of the model at some time during the time-history response. These peak values did not

Table 6.—UBC seismic design parameters

Code	Z	K	C	Base shear V = ZKCW	Period (T)	J
					Seconds	
1967 UBC	1.0	0.67	0.039	0.026 W	2.1	0.305
1970 UBC	1.0	.67	.039	.026 W	2.1	.45

necessarily occur simultaneously. They should be considered to represent only an envelope of maximum story shears.

The total response story shears were determined from the contribution of the first three modes of vibration, with the first-mode contribution accounting for much of the total response. Figures 20c and 21c also show the contributions of the second and third modes at the time of maximum story shears.

Maximum Overturning Moments

Envelopes of maximum overturning moments were calculated in a manner similar to that used in determining the maximum story shears. Figures 20d and

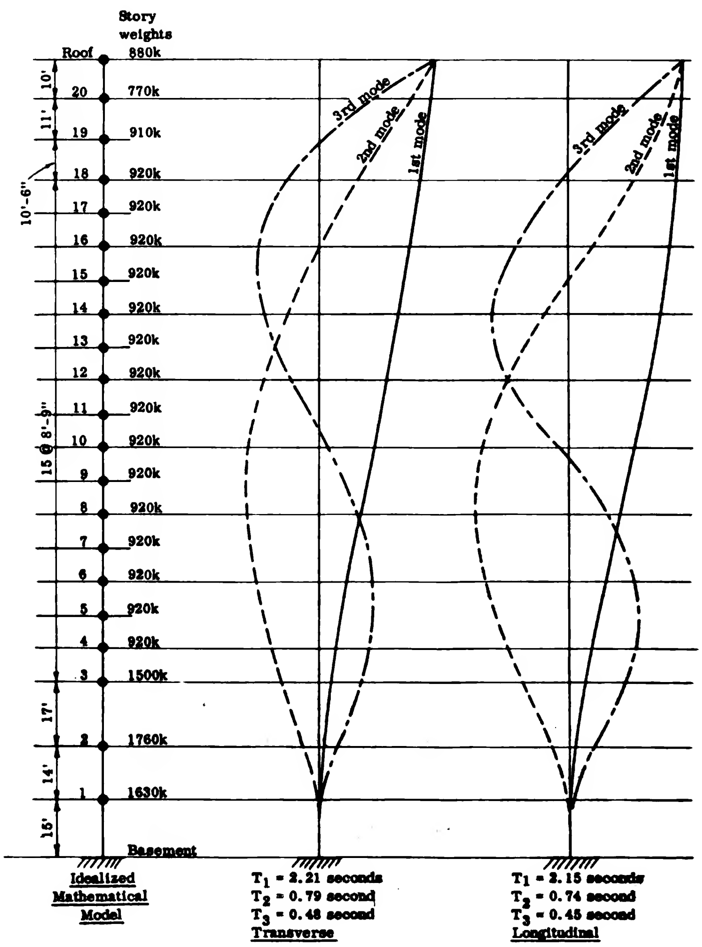

Figure 18.—Sheraton-Universal Hotel. Calculated periods and mode shapes.

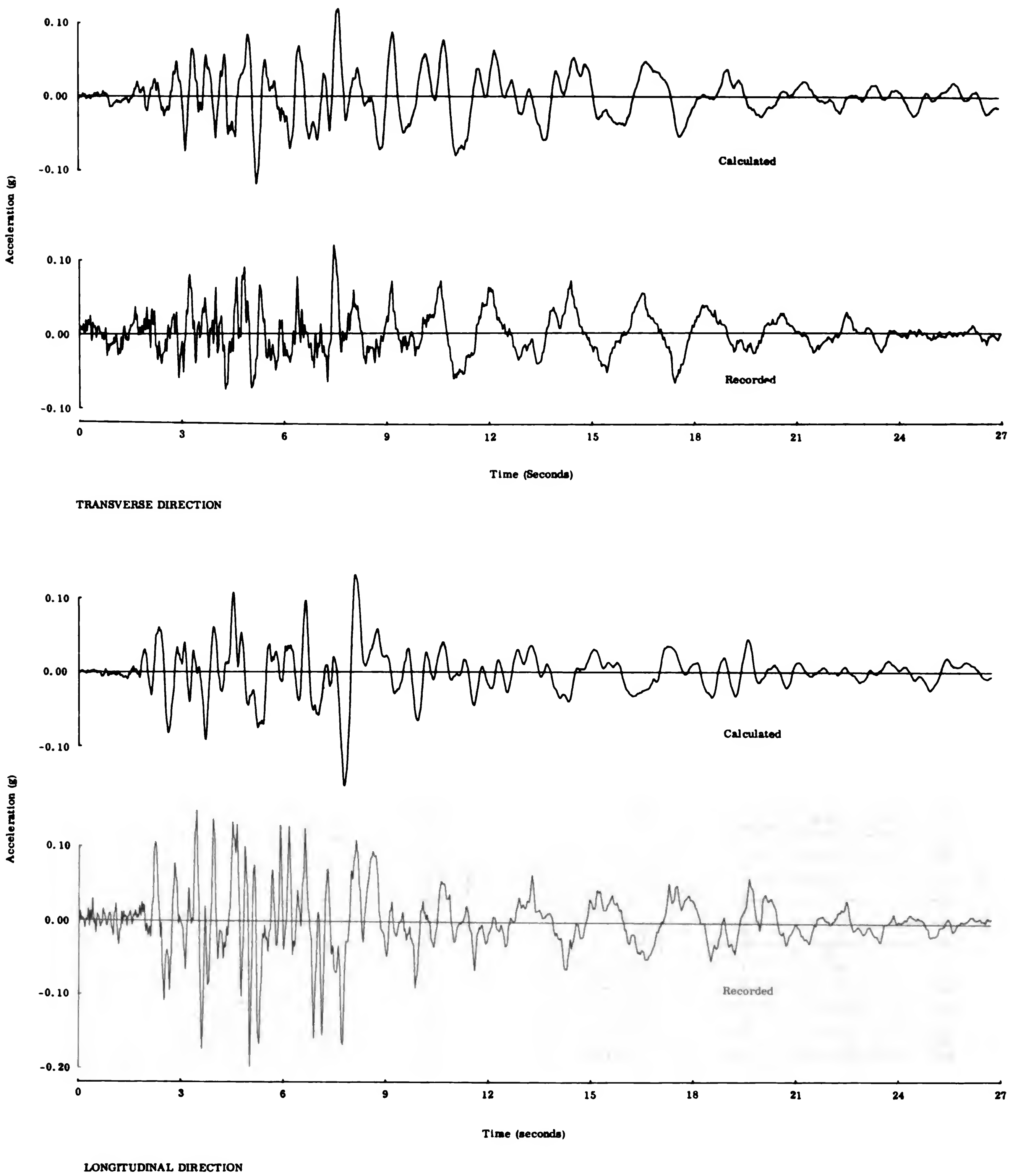

Figure 19.—Sheraton-Universal Hotel. Calculated and recorded accelerations at roof level.

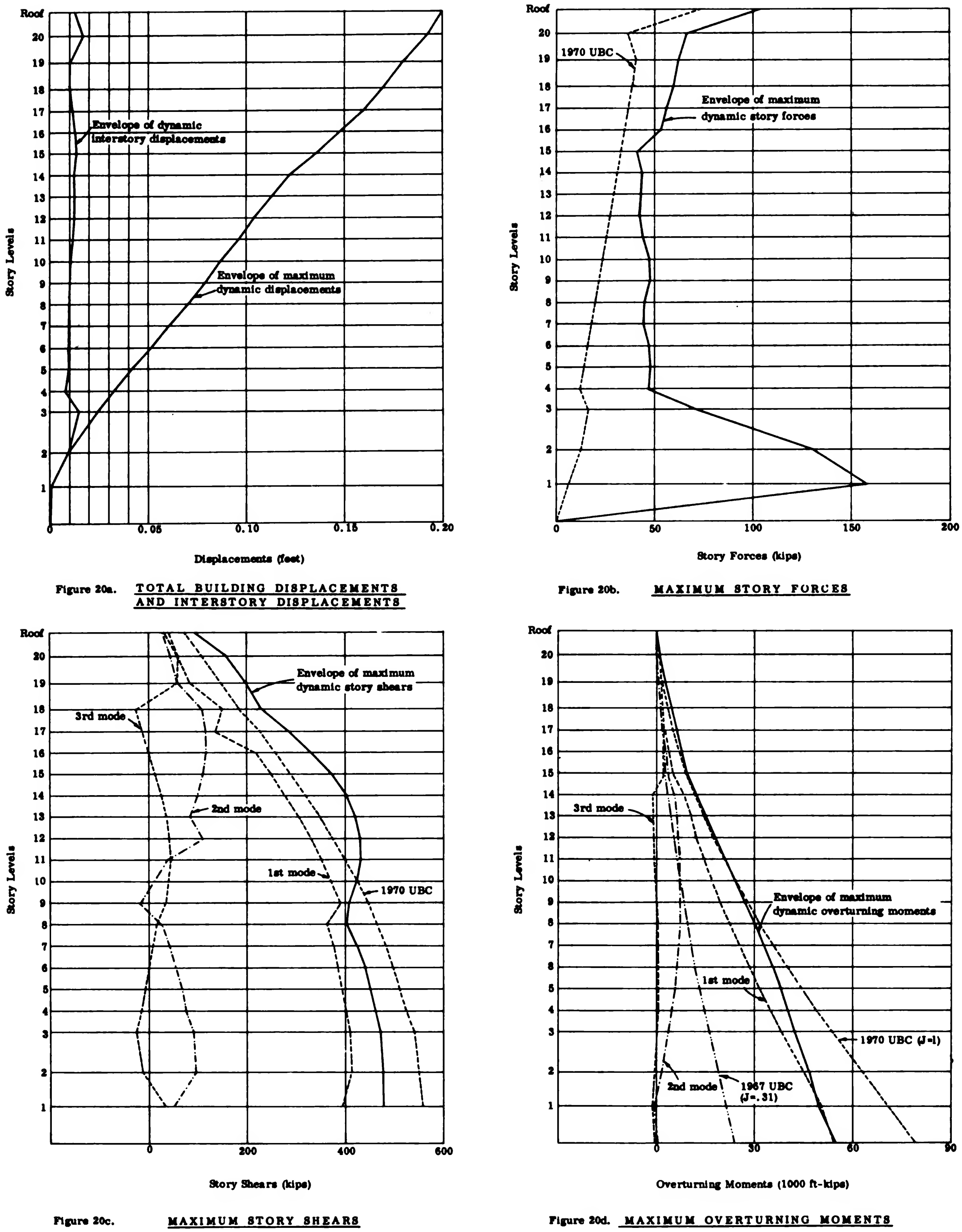

Figure 20.—Sheraton-Universal Hotel. Dynamic response and design code values for transverse (north-south) direction.

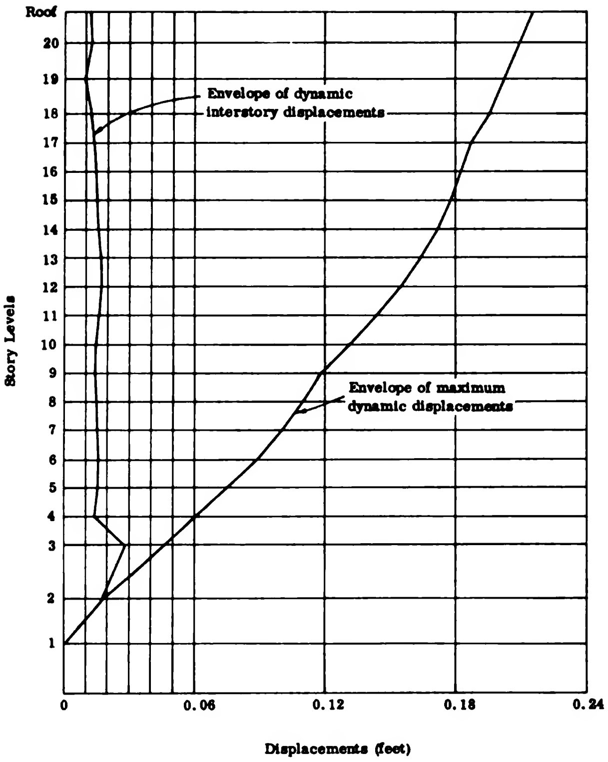

Figure 21a. TOTAL BUILDING DISPLACEMENTS AND INTERSTORY DISPLACEMENTS

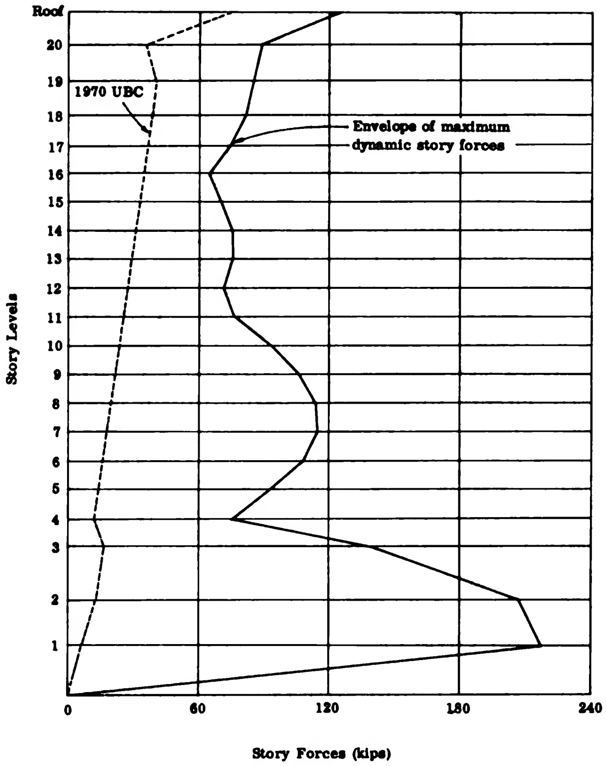

Figure 21b. MAXIMUM STORY FORCES

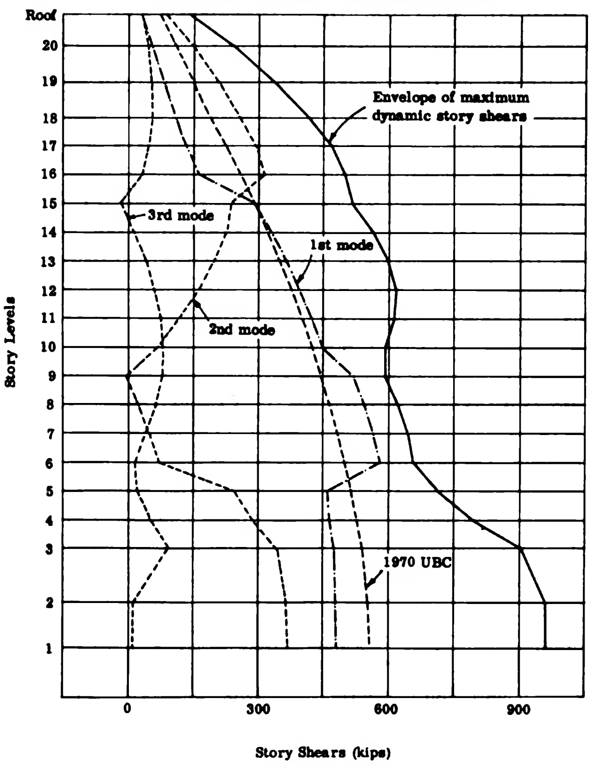

Figure 21c. MAXIMUM STORY SHEARS

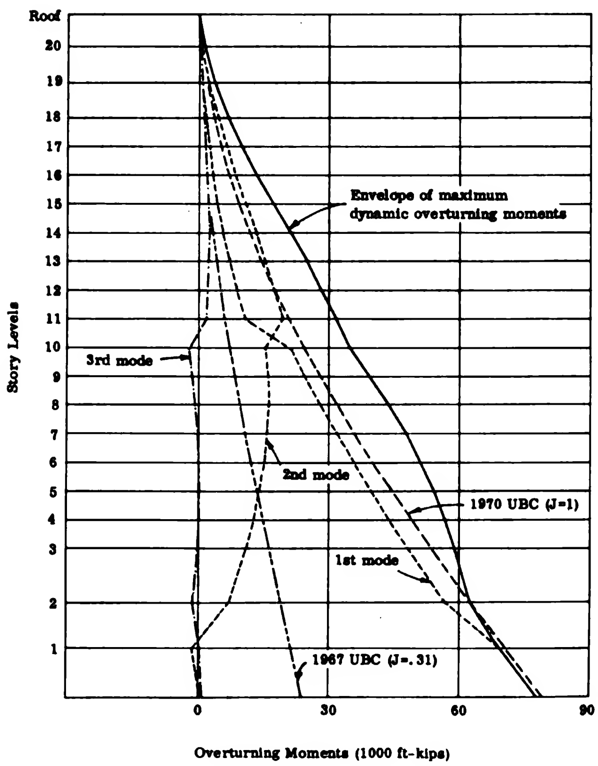

Figure 21d. MAXIMUM OVERTURNING MOMENTS

Figure 21.—Sheraton-Universal Hotel. Dynamic response and design code values for longitudinal (east-west) direction.

21d show these with corresponding 1967 and 1970 UBC values. The first three modes of vibration were determined as the maximum story shears were. Figures 20d and 21d also show the contributions of each mode to total response at the time of maximum response.

After publication of the 1970 UBC, the J factor for determining the base overturning moment was scrutinized considerably. Table 6 indicates that the 1970 UBC would require a minimum J value of 0.45, but subsequent amendments to the 1970 UBC have, in effect, increased the minimum J value to 1.0. For this reason, overturning moments determined with a value of $J = 1.0$ also have been included in figures 20d and 21d for a comparison with the 1967 UBC minimums in effect at the time the building was designed.

Loads on Key Girders

Earthquake loads on girders in the lateral force-resisting frames were investigated by comparing seismic and estimated vertical load effects to the estimated capacity for several representative key girders. Specifically, comparisons were made by calculating ratios of the controlling combinations of vertical and seismic moments to estimated ultimate moment capacities. These are indicated as M/M_{μ} in table 7. Ultimate moment capacities were computed on the basis of the recommendations of reference 3. Generally, girders were under-reinforced, indicating that the cross sectional area of available steel reinforcement, rather than crushing the concrete, limited ultimate moment capacity. In these calculations, a capacity reduction factor of $\phi = 1.0$, instead of the usual 0.9 design value, was used.

Results of the girder investigation (summarized in table 7) indicate that none of the girders experienced yielding of reinforcement and that no other failure conditions existed. Although the accuracy of the figures in table 7 is limited entirely by how well the modeling and analytical techniques approximate the physical situation, the low values of the results presented are considered good indices that yielding did not occur.

Shear capacities of the girders also were checked under the requirements of reference 3. In general, ultimate shear capacities were not exceeded by combined vertical and seismic loads.

Loads on Key Columns

Table 8 summarizes the combined vertical and seismic load effects on key columns. Ratios of critical vertical and seismic moments to estimated ultimate capacity, M/M_u have been calculated for each principal axis of bending. Values of summation of M/M_u greater than 1 would indicate that yield conditions probably had been approached or exceeded. Although highly dependent on the assumptions implicit in the analysis, the values are much less than 1. Consequently, the results presented in table 8 can be interpreted as indicating that the columns responded in an essentially elastic manner.

The recommendations of reference 3 guided the estimates of ultimate moment capacities, M_{μ}. Applied axial forces were considered. A capacity reduction factor of $\phi = 1.0$ was used for the rectangular tied columns.

The term P_{uo} in table 8 represents the ultimate axial load capacity of the column without bending moment present. Values of P/P_{uo}, where P is the ap-

Table 7.—Summary of girder M/M_{μ}

Beam B x D	Floor	Beam location between columns	M/M_u Top bars	Bottom bars
Inches				
16 by 18	1st	G–22 and H–22	0.33	0.31
16 by 18	2d	G–22 and H–22	.28	.37
16 by 18	3d	G–22 and H–22	.36	.41
16 by 12	4th to 8th	G–22 and H–22	.33	.39
16 by 12	9th to 13th	G–22 and H–22	.50	.86
16 by 12	14th to 18th	G–22 and H–22	.54	.55
12 by 16	19th	G–22 and H–22	.54	.20
12 by 20	2d	A–25 and A–28	.93	.17
12 by 36	3d	A–25 and A–28	.38	.22
18 by 12	4th to 13th	A–25 and A–28	.84	.11
18 by 12	14th to 18th	A–25 and A–28	.90	.14
26 by 33	19th	A–25 and A–28	.57	.23
26 by 12	Roof	A–25 and A–28	.62	.17

Table 8.—Summary of column interaction

Column	Size	Floor	Transverse direction $\frac{M_x}{M_{ux}}+\frac{M_y}{M_{uy}}$	$\frac{P}{P_{uo}}$	Longitudinal direction $\frac{M_x}{M_{ux}}+\frac{M_y}{M_{uy}}$	$\frac{P}{P_{uo}}$
	Inches					
A–20	22 by 22	2d	0.08	0.17	0.11	0.18
A–20	20 by 18	6th	.06	.22	.13	.20
A–20	20 by 18	11th	.09	.21	.14	.22
A–20	20 by 18	20th	.50	.03	.27	.03
A–20	20 by 18	Roof	.18	.01	.15	.01
B–22	22 by 22	2d	.10	.45	.18	.44
B–22	20 by 20	6th	.09	.50	.15	.48
B–22	20 by 20	14th	.18	.24	.23	.25
B–22	12 by 20	17th	.39	.22	.36	.22
B–22	12 by 20	18th	.38	.16	.50	.19
B–22	12 by 20	19th	.34	.10	.20	.17

plied load, are given to provide an approximate index of axial load effects. The summation of M/M_u values (for biaxial bending) is also an approximation of a more complicated interaction relationship.

DISCUSSION AND INTERPRETATION OF RESULTS

Comparison of Calculated Versus Code Forces

The previous paragraphs presented results of the dynamic analysis and subsequent comparisons of those results with code values. The results of the dynamic analysis showed that, generally, the level of code seismic forces was less than what the structure was required to resist. Examination of the computed member forces in the moment-resisting frames revealed that member loads, due to combined vertical and seismic forces, were less than those expected to cause yielding in the reinforcement. No member of the lateral force-resisting system was determined to have suffered structural damage.

Modal Analysis Procedures

In order to test the accuracy of the mathematical models, an attempt was made to verify both mode shapes and periods. Unfortunately, mode shapes could not be confirmed by the recorded motion because of the lack of sufficient data. Records of motion were obtained for only the roof level, and these did not provide a sufficient number of data points to verify calculated mode shapes.

However, results were better when calculated periods were compared with recorded building periods. This comparison helped to determine the accuracy of the mathematical models. Since any one of several parameters could be changed to improve the correlation between calculated and measured periods, calculating the correct period did not assure absolutely that the mathematical model truly represented the actual structure. Verification of computed mode shapes using data obtained from building-motion records at several intermediate levels would have improved the confidence level of the mathematical models.

Comparison of Recorded and Computed Responses

Comparing acceleration time histories for the roof level yielded comparisons of recorded and computed responses. The general shape of the computed time history of the acceleration response at the roof correlates reasonably accurately with the recorded values in figure 19.

Calculated fundamental periods for the structure were approximately 2.1 seconds in the longitudinal direction and 2.2 seconds in the transverse direction. The apparent recorded fundamental period of the structure during the first 6 seconds of the earthquake was much less than 2.2 seconds in the transverse direction. It was somewhat less than the calculated 2.1-second value in the longitudinal direction. The calculated periods were obtained from the mathematical models of the bare structural frames.

The shorter fundamental period in the transverse direction for the first few seconds of motion suggested that architectural elements initially provided the structure with additional horizontal rigidity. Guest rooms have a considerable number of party walls running in the transverse direction. While the architectural drawings show these walls to be seismically separate from the lateral force-resisting system, they may have resisted lateral force during the first part of the earthquake, at least until friction was overcome.

After 6 seconds of motion, the apparent period of the structure was approximately 2.1 seconds in both directions. This indicated that, after this time, enough force had been generated to overcome any bond between structural and nonstructural elements. Only the bare structural frames resisted the earthquake motion.

Table 5 lists computed and recorded maximum accelerations, displacements, and interstory drifts. Computed and recorded accelerations accurately correspond for the transverse direction. In the longitudinal direction, the recorded value surpasses the computed value by about 30 percent. Although the general shape and character of the computed and recorded plots of accelerations reasonably agree, correlation of the amplitudes possibly could be improved by varying applied damping values for the different modes, rather than by applying the same damping value to all modes.

Correlation With Damage Observations

The dynamic analysis indicated that no structural damage should have occurred from the earthquake. The field damage survey verified this. Damage occurred exclusively to architectural elements, totaling only $2,100.

Vertical Accelerations

Vertical motion recorded at the roof showed a considerable amplification over that recorded at the basement level. It also recorded a larger amplitude than either of the two horizontal directions. Interestingly, the opposite situation existed at the basement level, with vertical motion showing less amplitude than either of the two horizontal directions.

Figures 11 and 12 show the strong vertical amplification effects of the building, stronger than either the roof or basement experienced in the horizontal directions. This increase in vertical response, however, occurred with vertical motion having a predominantly higher frequency content than that of the horizontal components.

SUMMARY OF FINDINGS

As a result of the dynamic analysis of the Sheraton-Universal Hotel for effects of the San Fernando earthquake, including a comparison of calculated dynamic versus code seismic forces, these conclusions have been drawn.

1 The building responded to the earthquake in an essentially linear-elastic manner, but with an equivalent viscous damping of approximately 10 percent of critical damping.

2 Calculated earthquake forces generally were greater than prescribed code minimums.

3 Damage was limited essentially to architectural elements, such as seismic joint covers at the junction of the tower structure and the low wings. There was apparently no structural damage other than minor cracking in two locations.

4 Calculated building response indicated that no major structural damage would be expected. This was verified by field observations.

RECOMMENDATIONS

This study recommends several areas of possible change in seismic design and instrumentation.

Seismic Requirements for Architectural Elements

Expansion joints, flashings, partitions, and stairwells should be designed for seismic movements. The amount of movement to be designed into these elements should be based upon maximum possible interstory drifts, rather than upon deflections computed for code seismic forces (unless code seismic forces are increased considerably).

Strong-Motion Instrumentation

Strong-motion recording devices should be placed to provide a record of true transverse and longitudinal motion without any undue contributions from real or accidental torsional effects. Vertical records should be taken at or near a column to avoid local, possibly anomalous, effects from more flexible elements such as thin slabs.

Verification of mode shapes could be improved if a greater number of recording instruments were placed between the top and ground levels of highrise buildings. By placing only one intermediate or midheight instrument, verification may be doubtful.

Bank of California (28)

15250 Ventura Boulevard, Los Angeles

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JOHN A. BLUME & ASSOCIATES, ENGINEERS
San Francisco, Calif.

DESCRIPTION OF BUILDING

The Bank of California Building is a 12-story reinforced concrete structure located in the Sherman Oaks district of Los Angeles. This area is some 17 miles south of the epicenter, near the southern end of the San Fernando Valley. Immediately south of the site is the easternmost portion of the Santa Monica Mountains.

The structure, completed in 1970 at a cost of $4 million, currently functions as an office building. Plan dimensions of the floors are 60 by 161 feet, except at the first story where plan dimensions are 90 by 161 feet. Story heights are typically 13 feet, except at the first story where a 16-foot height is used. The main roof stands 159 feet above the ground floor. A mechanical penthouse occupies some 30 percent of the main roof area. Figures 1 and 2 provide views of the east and north elevations of the building.

Soil conditions at the site are primarily silt and silty sand with lesser deposits of clay and sand. A typical soil boring log is shown in figure 3 (reference 8). Because the upper soils at the site are only moderately firm and would tend to become weaker and more compressible when wet, pile foundations were provided. These are shown on the foundation plan in figure 4, which also includes a plot plan indicating the location of the typical soil boring. Piles shown on the foundation plan are drilled and cast-in-place concrete piles 35 to 50 feet long.

The structure was designed in 1969 under the requirements of the 1968 Los Angeles City Building Code. Except at the lower levels where some shear walls are located, lateral forces in each direction are resisted by moment-resisting reinforced concrete space frames consisting of columns and girders. In the transverse (east-west) direction, moment-resisting frames (figs. 5, 6, 7, 8, and 9) in the exterior bays along column lines A and I extend the full height of

Figure 1.—Bank of California. East elevation. John A. Blume & Associates photograph.

Figure 2.—Bank of California. North elevation. John A. Blume & Associates photograph.

the structure, whereas interior frames extend only from the ground to the third floor along column lines B, D, E, F, and H.

Above the third floor, interior columns continue to the roof as indicated in figure 8. The beams framing into these columns are merely wide joists, with reinforcement designed to carry only vertical loads; consequently, these frames do not really function as full-height lateral load-carrying frames. Similarly, in the longitudinal (north-south) direction, exterior frames along column lines 1 and 3 are reinforced to carry the design lateral forces. The interior frame on column line 2 was designed to carry only vertical loads.

The longitudinal frame along column line 1 and the transverse frames along column lines A and I have their spandrel beams set back approximately 3 feet from the column line. Along column line 3, columns between the ground floor and the second floor have been omitted at alternate columns between lines C and H.

The moment-resisting frame elements included shear walls below the third-floor level in the longitudinal direction. Two shear walls, each two stories high, rise along column line 1. A one-story-high separation wall rises along column line 4, adjacent to the property line. It serves as a shear wall. Figure 5 shows these walls.

Typical floor construction consists of a $4\frac{1}{2}$-inch slab on a 17-inch-deep pan-joist system, which spans from girder to girder. All floor and girder construction consists of lightweight concrete. Rectangular tied columns, constructed with regular weight stone aggregate concrete, carry the girders. Table 1 indicates the properties of materials used in construction.

Permanent nonstructural elements, consisting of gypsum wallboard and metal stud partitions, enclose the elevator shafts, stairwells, duct-shafts, and toilet

rooms. However, at the time of the earthquake, the interior partition construction was incomplete.

Typical enclosure walls around the perimeter of the building consist of 2½-foot-high metal stud walls, except at the third floor where a concrete enclosure wall was used. A continuous curtain wall between columns stands on top of these walls. The mechanical penthouse construction consists of concrete masonry block walls with a metal deck and a steel beam roof system.

During construction, the structural engineer followed standard inspection procedures. He assumed full responsibility for interpretation of the structural drawings and for periodic inspections. A full-time deputy building inspector also was provided, as well as a part-time city inspector, as required by the Los Angeles City Department of Building and Safety.

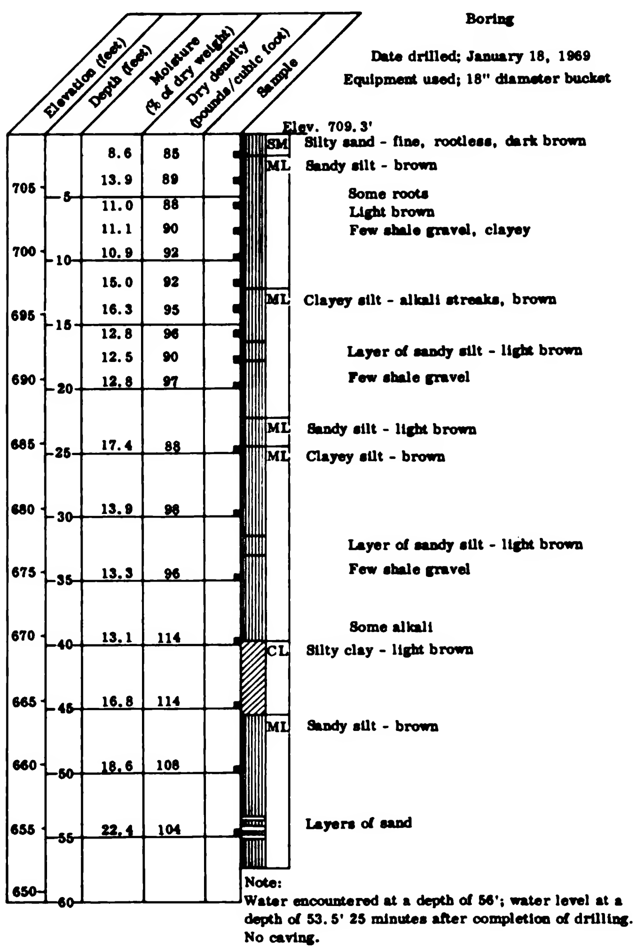


Figure 3.—Bank of California. Log of typical soil boring.

Table 1.—Properties of construction materials

Concrete

Location in structure	Aggregates	Unit weight	Minimum specified compressive strength (f'_c)	Modulus of elasticity (E)
		pcf [1]	*psi* [2]	*psi* [2]
Columns.........	Regular weight (ASTM C-33)	150	4,000	3.9×10^6
Beams, joists, and slabs.	Lightweight (ASTM C-330)	110	3,000	2.1×10^6
Foundation.......	Regular weight (ASTM C-33)	150	3,000	3.3×10^6

Reinforcing steel

Location in structure	Grade	Minimum specified yield strength (f_y)	Modulus of elasticity (E)
		ksi [3]	*psi* [2]
Walls, slabs, ties, and stirrups.	Intermediate (ASTM A-615, Grade 40)	40	29×10^6
All other...........	High-strength (ASTM A-615, Grade 60)	60	29×10^6

[1] Pounds per cubic foot.
[2] Pounds per square inch.
[3] Kips per square inch.

EARTHQUAKE DAMAGE

The Bank of California experienced both structural and nonstructural damage. Structural damage consisted of cracking and spalling of concrete columns, spandrels, girder stubs, and a parapet wall at the first story. Nonstructural damage resulted to partitions, ceilings, stairwells, stairs, mechanical equipment, and some furnishings.

All of the damage experienced was reparable. Repairs totaled **$44,000; $12,000** was spent on epoxy repair of damaged concrete elements.

Structural Damage

Generally, visible structural damage was slight and consisted almost entirely of minor cracking and spalling of concrete (figs. 10 and 11). However, in some areas, extensive cracking occurred, particularly at the exposed girder stubs at the exterior columns along the second-floor level (figs. 12 and 13).

At the penthouse level, slight spalling of patched concrete was observed where the exterior penthouse walls joined the penthouse columns. Between the sixth and 11th floors, there was some minor spalling

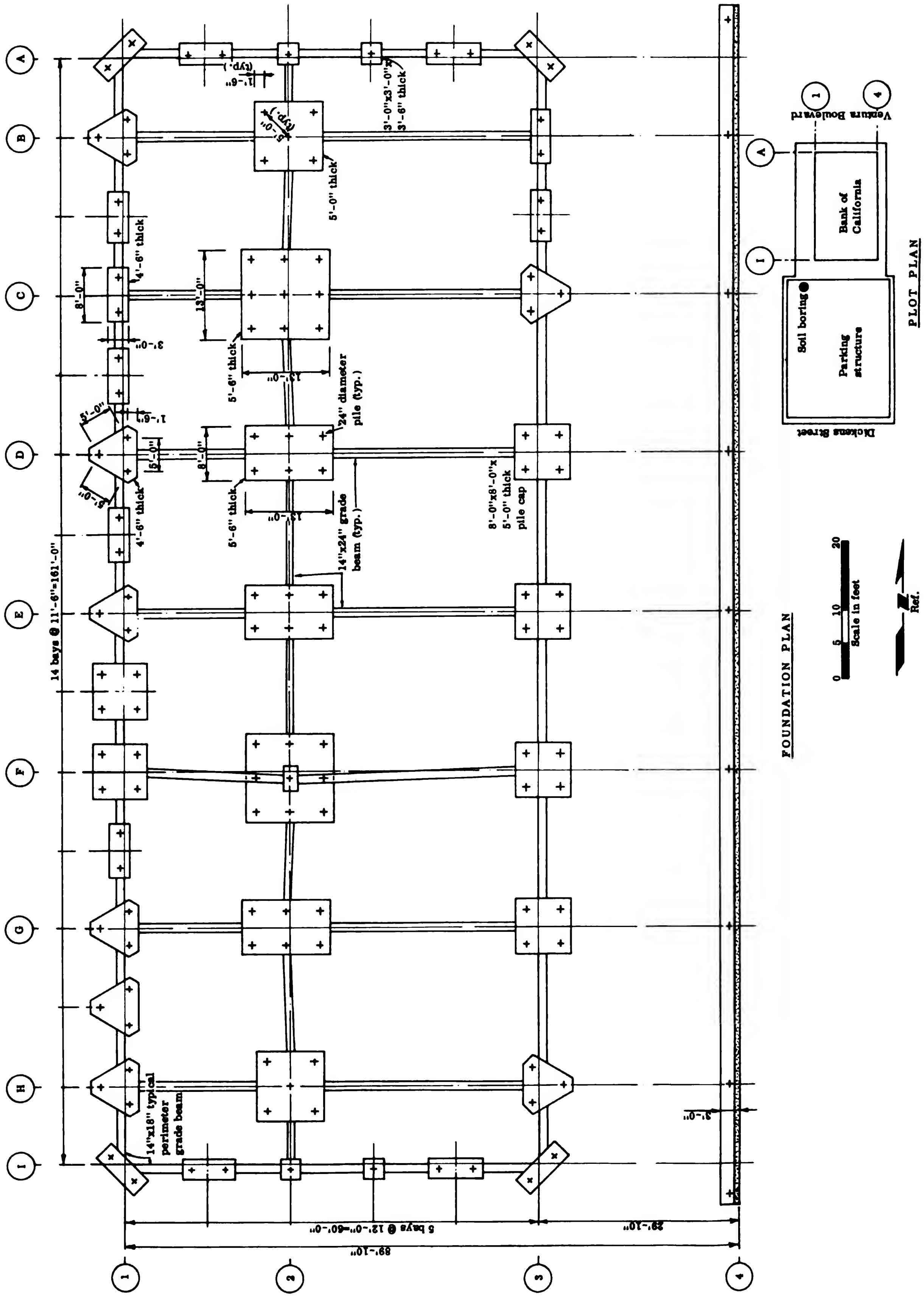


Figure 4.—Bank of California. Foundation plan.

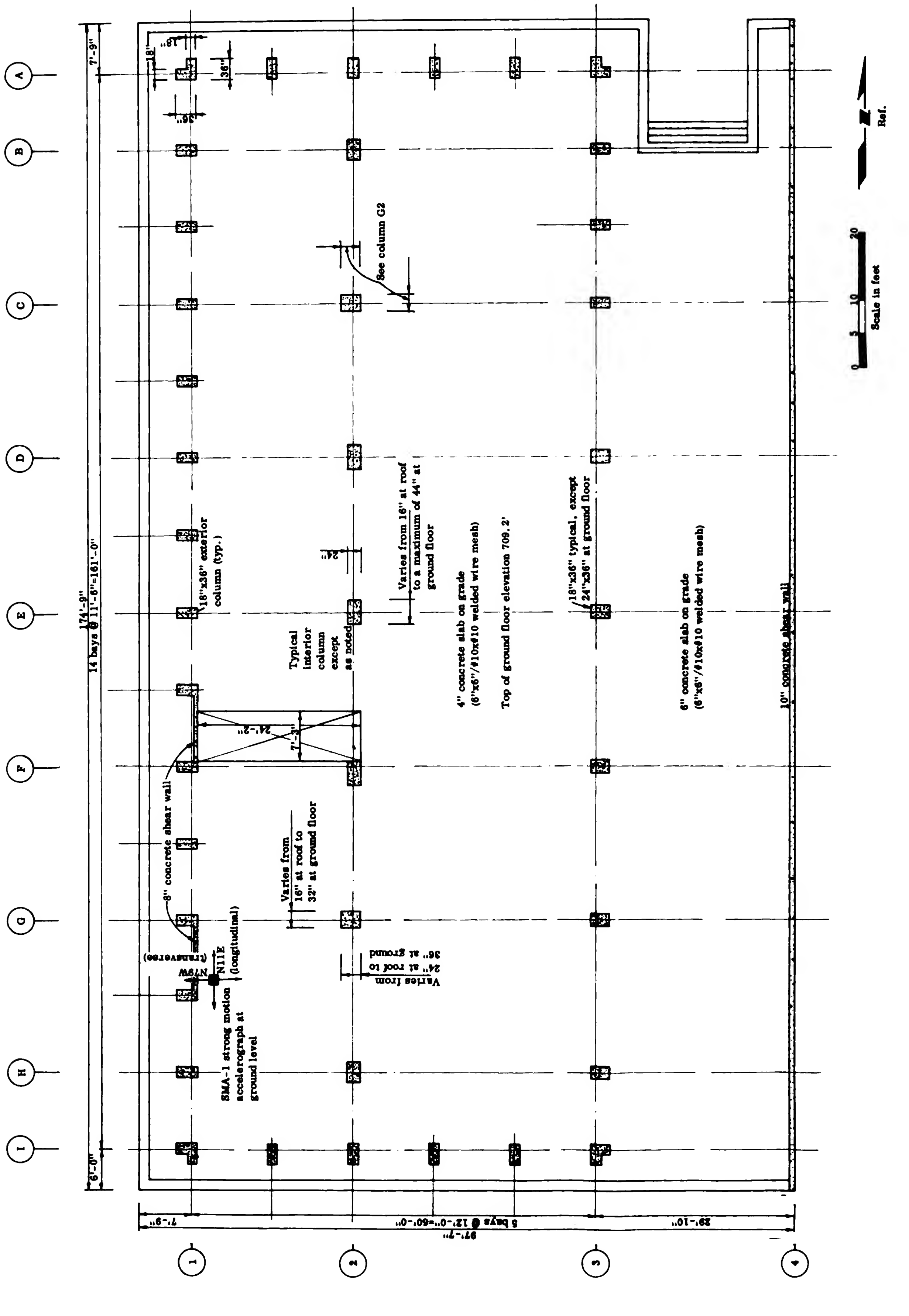


Figure 5.—Bank of California. Ground-floor plan.

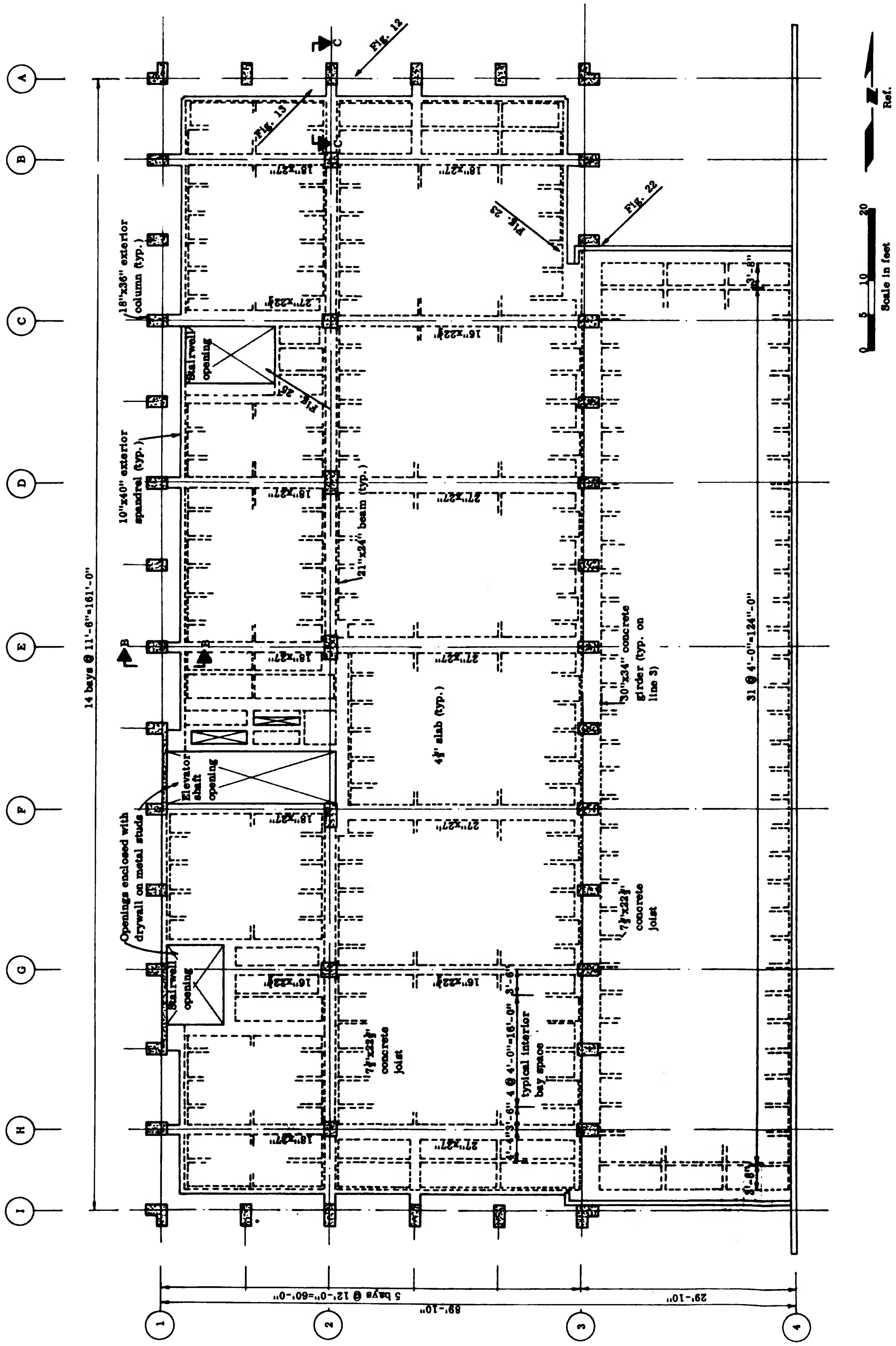


Figure 6.—Bank of California. Second-floor plan.

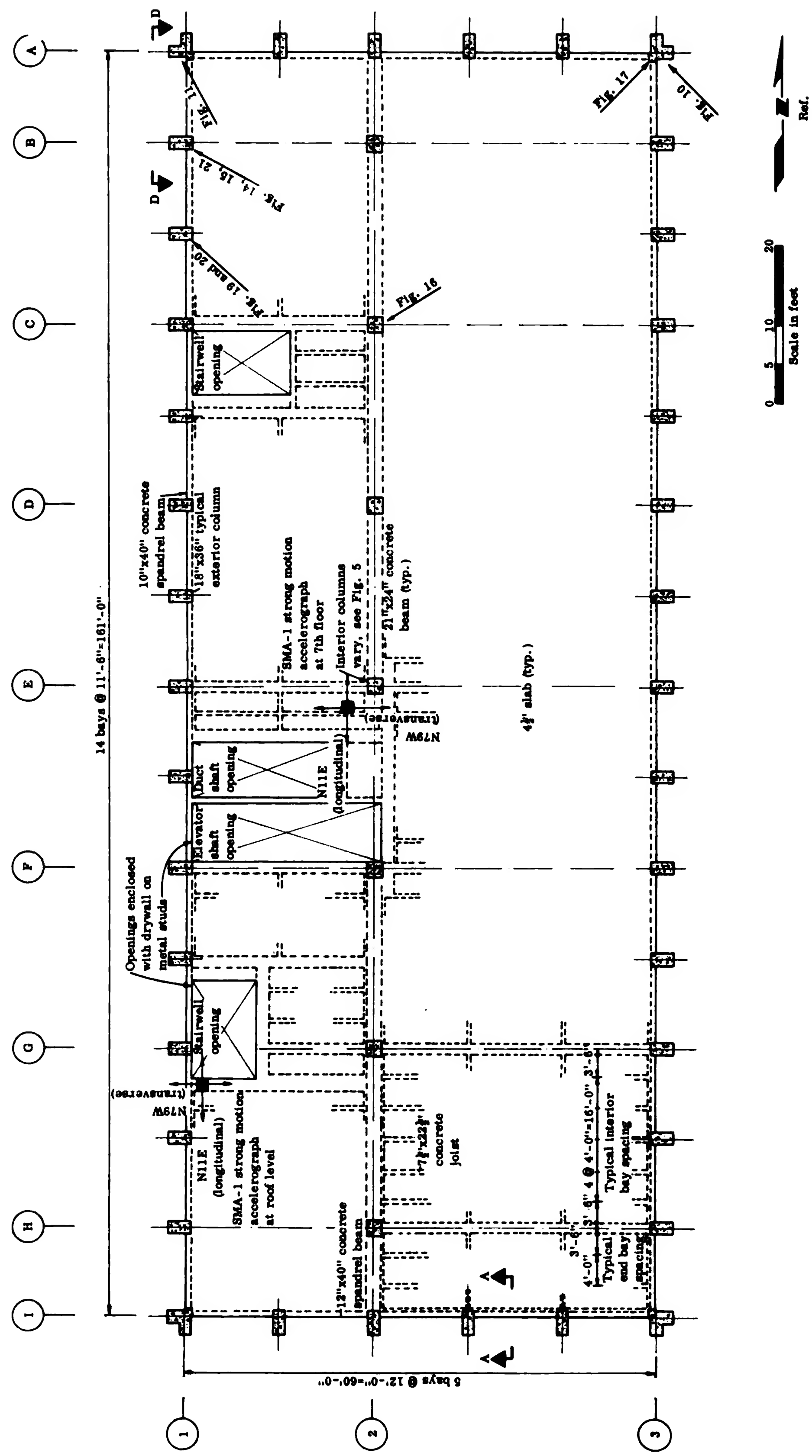


Figure 7.—Bank of California. Typical floor plan, third through 12th floors (roof similar).

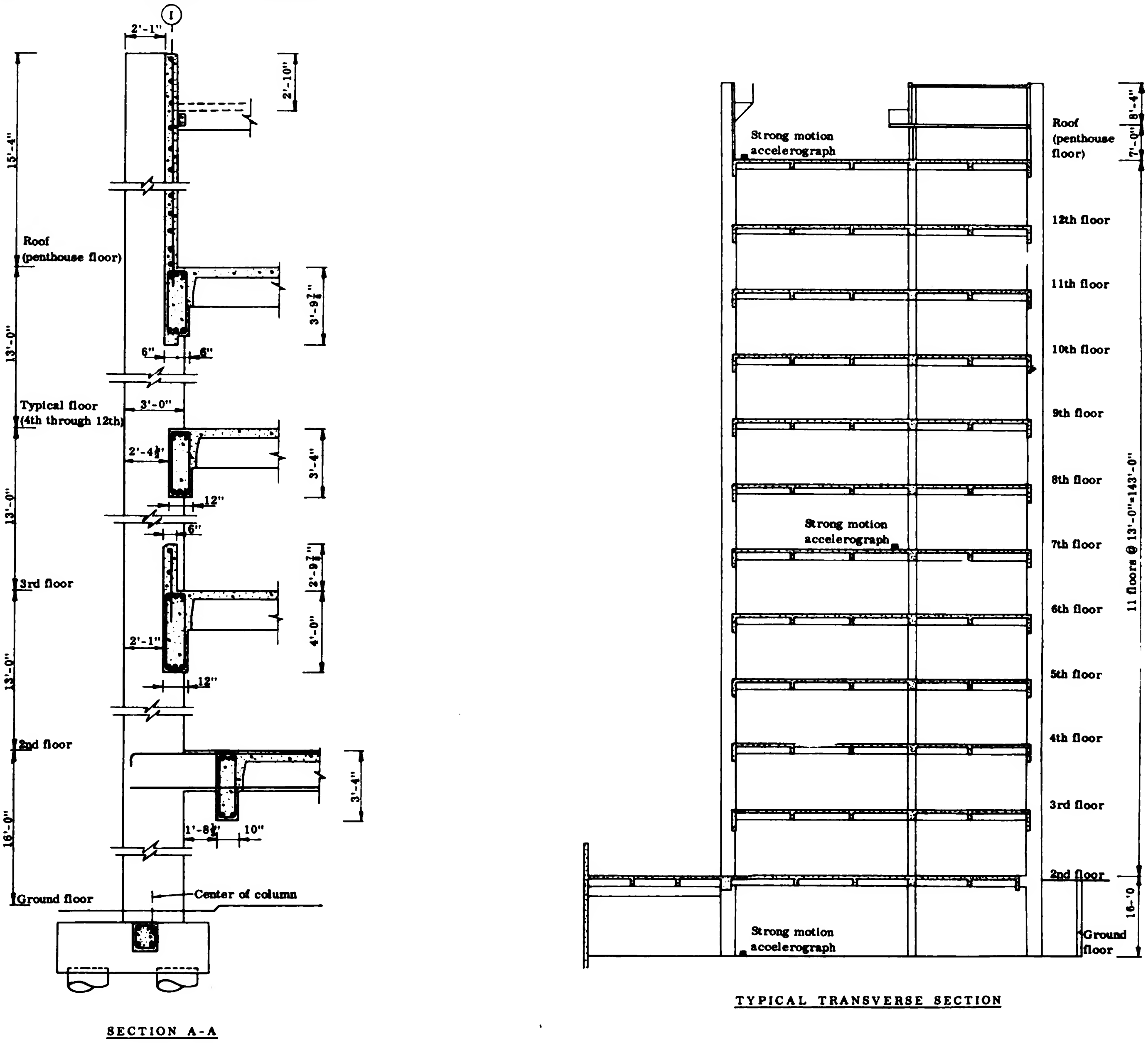


Figure 8.—Bank of California. Typical transverse section and structural details.

of some east and west columns. This was at the floorline on the inside face of the building.

At floor level on the third, fourth, and fifth floors, exterior columns typically spalled (figs. 14 and 15). On the east and west sides of the building, spalling also occurred at the top of the columns, but only on the interior column face. On the east and west sides, some spandrels showed areas of spalling near the columns.

At these same levels, interior columns experienced no damage, but a series of cracks was observed in the floor slab around columns (fig. 16). Corner columns revealed some hairline cracks on a slight diagonal extending in the east-west direction (fig. 17).

Horizontal hairline cracks were observed at third-floor spandrel beams along construction joints. Some of these cracks extended through the columns.

The spandrel-column detail shown in figure 18 facilitated construction (concrete pours) from the fourth floor to the roof. Major joint rotation and column or spandrel spalling coincided with areas where this detail was used (figs. 19, 20, and 21).

The typical floor plan changes at the second floor. Some of the exterior columns stop, not extending to the first floor below, and a one-story low roof structure is attached at the east side by means of a para-

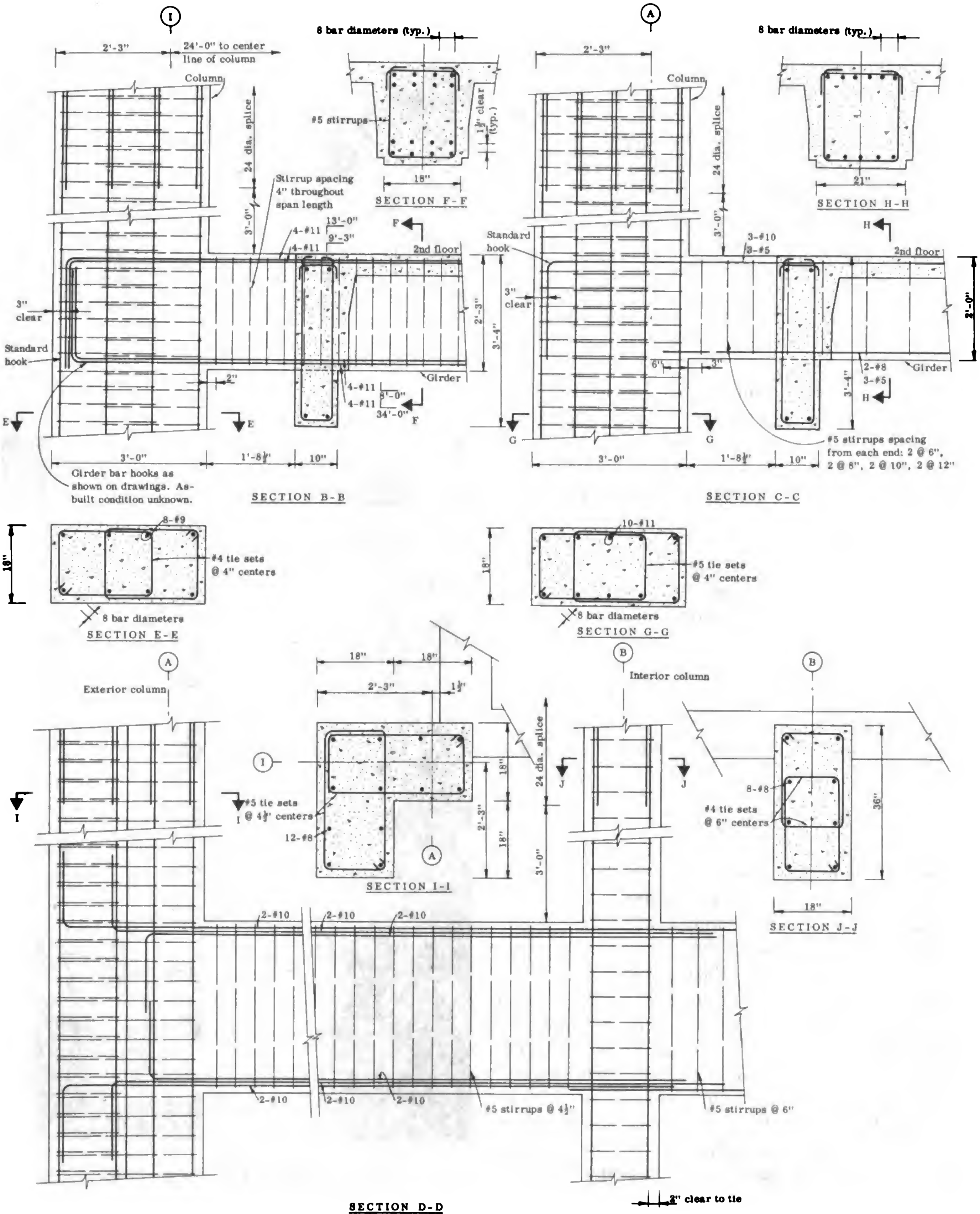


Figure 9.—Bank of California. Typical details of reinforcement at column-girder joints.

pet (fig. 22). The north, south, and partial west floor edges do not extend to exterior columns at this level. The west side edge ties to two shear walls extending to the third floor (fig. 6).

At the second floor, girders projecting beyond the spandrel line were damaged. The parapet on the one-story low roof structure attached to the main building also was damaged. The torsion induced in girder stubs projecting from columns on the north and south sides caused cracking as shown in figures 12 and 13. The stubs are located at alternate columns, except at corners, and for the north and south sides were not considered seismic force-resisting elements. Girder stubs on the west side also showed some bending cracks.

A cold joint bonded the low roof structure to the side of an east building column. This joint, not designed to be integral, sheared free during the earthquake (figs. 22 and 23). The east wall of the one-story portion also showed horizontal cracks at construction joints, but no other damage or shear cracks were seen.

No noticeable foundation distress, settlement, or overturning effects were observed. The cast-in-place and drilled pile foundation seemed to perform satisfactorily.

Nonstructural Damage

Nonstructural damage was distributed extensively between the sixth and 11th floors. Partitions running in an east-west direction pulled away from east and west exterior columns as shown in figure 24. Thus, ceiling tiles fell out. Between the third and fifth floors, some bolts on east-west spanning stairs were pulled from concrete floor beams. Partition cracking also occurred in the stairwells at these levels (fig. 25), decreasing below the fourth floor. At the second-floor level, windows shifted in the mullions on the north and south sides of the building.

The east-west running gypsum wallboard on steel stud partitions was fastened to both the floor and suspended ceiling. As the building displaced laterally during the earthquake, racking of the partitions and shortening of the hung ceiling took place. As illustrated in figure 26, the ceiling level shortened as much as ¾ inch. This indicated story drifts of at least ¾ inch. Such partition and ceiling displacements were observed at the fourth, fifth, and sixth floors, and to a lesser extent at the seventh, eighth, and ninth floors.

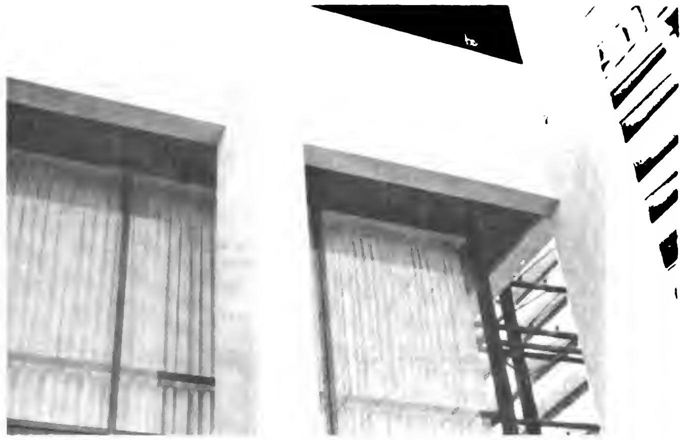


Figure 10.—Bank of California. Spalling of paint surface at northeast corner column (third-floor) cold joint. Only hairline cracks were observed in the joint area. Brandow & Johnston photograph.

Figure 11.—Bank of California. Spalling of concrete at northwest corner column, fourth floor. This condition, which did not occur at any other location, may have resulted from a defective concrete patch during construction. Brandow & Johnston photograph.

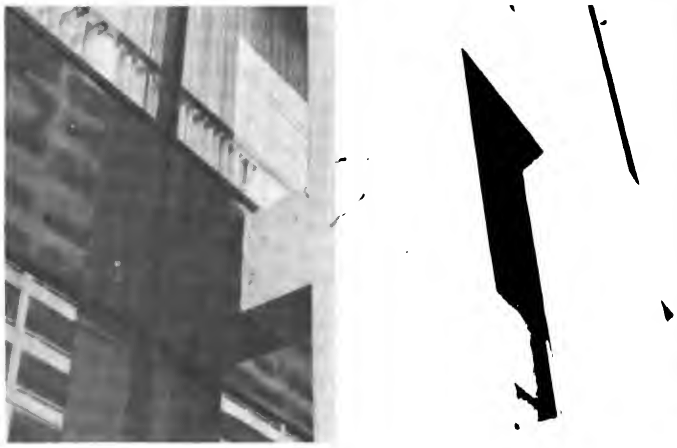


Figure 12.—Bank of California. Exterior second-floor girder-column joint in nonseismic north-south frame. Note that spandrel beam, which lies hidden behind curtain wall, is considerably offset from column centerline. John A. Blume & Associates photograph.

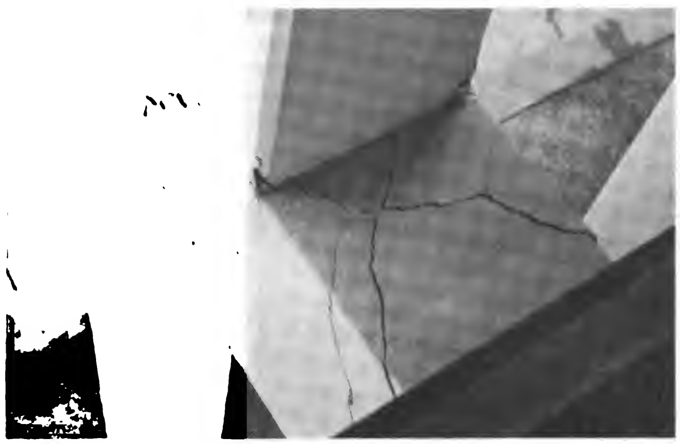


Figure 13.—Bank of California. Top view of second-floor girder-column connection shown in figure 12. Evidence of torsional cracking resulting from offset spandrels is apparent. Brandow & Johnston photograph.

Figure 14.—Bank of California. Spalled area on interior face of exterior column. Column reinforcing has been further exposed by chipping. Brandow & Johnston photograph.

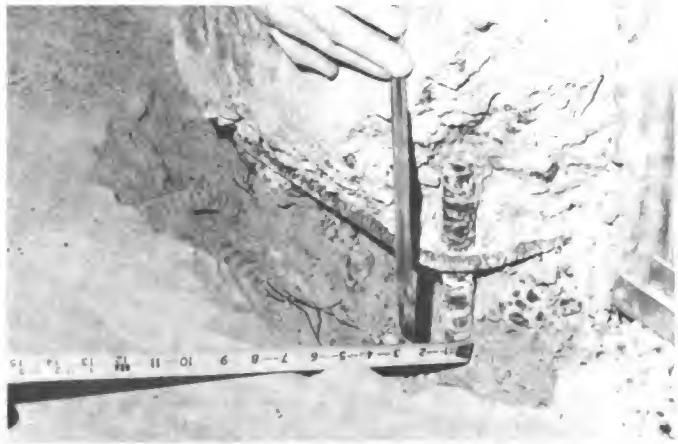


Figure 15.—Bank of California. Detail of spalled and chipped area similar to that shown in figure 14. Note that top joist reinforcement was terminated by hooks at column face and was not extended into confined area of column. Brandow & Johnston photograph.

Figure 16.—Bank of California. Type of floor and girder cracking found near second interior column, in from north end of the building on both fourth and fifth floors. Brandow & Johnston photograph.

Figure 17.—Bank of California. Diagonal cracks at north end of fourth floor near corner column. Brandow & Johnston photograph.

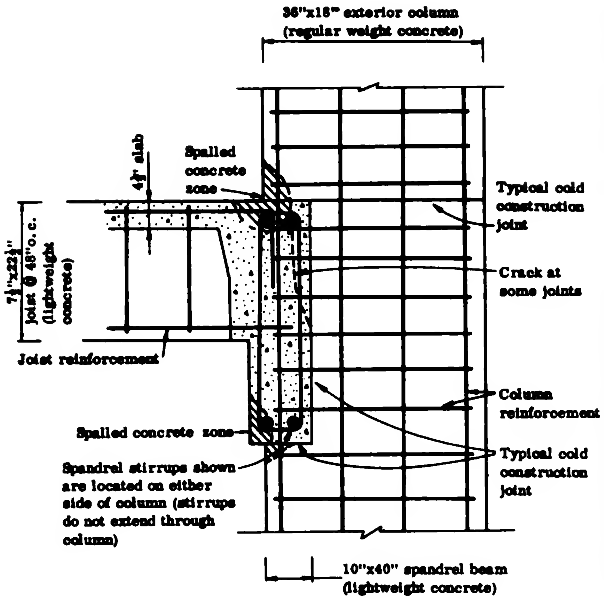


Figure 18.—Bank of California. Typical construction of exterior column-spandrel-joist connection from fourth floor to roof.

Figure 19.—Bank of California. Spalled lightweight concrete on fifth-floor spandrel. Note also chipped concrete on left side of column. Brandow & Johnston photograph.

Figure 20.—Bank of California. Spalled concrete in joint zone of spandrel and exterior column. Spandrel is offset from column centerline. Little flexural cracking was evident in members on either side of joint. Note main bottom reinforcing without confining stirrups within column zone. Brandow & Johnston photograph.

Figure 21.—Bank of California. Spalled area of spandrel beams after all cracked and loose concrete had been chipped away in preparation for epoxy repairs. Prior to this photograph, epoxy pressure grouting was already under way with an epoxy sand mortar injected at a pressure of 20 psi. Brandow & Johnston photograph.

Figure 22.—Bank of California. Low roof connection to office tower columns at second-floor level. Original building plans specified a seismic separation at this point, but this detail was poured monolithicly during construction. Brandow & Johnston photograph.

Nonstructural damage also occurred to mechanical equipment and building contents. Between the sixth and 11th floors, potted plants toppled. Water bottles toppled from their bases at the third and eighth floors. In the mechanical penthouse, a compressor mounted on an inertia block came off its spring mounting.

A cooling tower basin spilled water. The cooling tower was supported on steel beams spanning in a north-south direction. No tilting was observed, but a sheet metal channel support for the cooling tower appears to have buckled locally. Chiller units, loosely attached to a housekeeping pad, slightly twisted about a vertical axis.

RECORDED EARTHQUAKE RESPONSE

Earthquake motions in the building were recorded by Kinemetrics-type SMA–1 strong-motion accelero-

Figure 23.—Bank of California. Closeup detail of figure 22. Brandow & Johnston photograph.

Figure 24.—Bank of California. Cracking in drywall partitions provided an indication of interstory deflections. This east-west running partition separated substantially from the east exterior curtain wall. Brandow & Johnston photograph.

graphs. These were located on the roof, seventh floor, and ground floor (figs. 5 and 7).

At each level, accelerations were recorded along the vertical and both horizontal axes of the building. Approximately 28 seconds of motion was recorded. Table 2 lists peak recorded accelerations. Figures 27, 28, and 29 show plots of the digitized recorded accelerations.

Table 2.—Peak recorded accelerations

Station	Longitudinal (N.11°E.) component	Transverse (N.79°W.) component	Vertical component
	g	*g*	*g*
Roof.............	0.277	0.188	0.150
7th floor..........	.262	.255	.172
Ground floor......	.230	.155	.108

Digitized records of the recorded ground-level motion determined response spectra for 2 and 10 percent of critical damping through use of the SMIS-4 program. Figures 30, 31, and 32 show these for the transverse, longitudinal, and vertical components of motion. The spectra have been plotted on four-way log paper to aid the reading of either pseudo-absolute acceleration, pseudo-relative velocity, or relative displacement values.

Figure 25.—Bank of California. Side walls of northwest stairwell showed signs of "working" during the earthquake. Brandow & Johnston photograph.

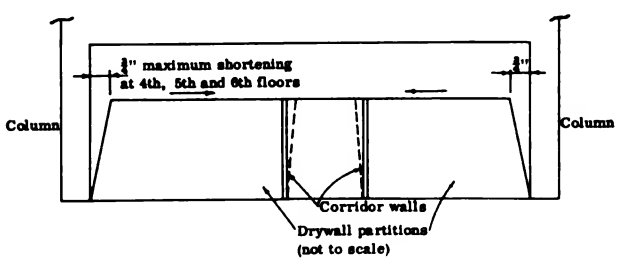


Figure 26.—Bank of California. Shortening of transverse drywall.

MATHEMATICAL MODELING

In the preceding general discussion of analytical procedures, mathematical models of the physical characteristics of the structure in its response to earthquake ground motion were described. Member stiffness properties of moment of inertia, shear area, and cross sectional area were determined for each element of the lateral force-resisting system.

These properties, with the appropriate elastic moduli (table 1), were then used to determine the initial stiffness characteristics of the entire lateral force-resisting system in each principal direction. For each member, stiffness properties were assumed to be those indicated in table 3. Values of the gross moment of inertia, I_o, of each girder were adjusted to compensate for the effects of integrally cast slabs, reinforcement, and the effective member lengths, which actually were shorter than the centerline to centerline distances used in the analyses.

Table 3.—Member stiffness properties

Member	Moment of inertia	Shear area	Cross sectional area
Girder	$2I_o$	......	......
Column	I_o	$5/6\ A_g$	A_g
Shear wall on column line 4	I_o	$5/6\ A_g$	A_g
Shear walls on column line 1 [1]	......	......	......

[1] The two shear walls on line 1, which extend from the ground floor to the third floor, were modeled as diagonal struts with the same horizontal stiffness characteristics as the shear walls.

Two-dimensional mathematical models were developed because this building report was limited to planar analyses. In effect, the analyses assumed the building to be symmetrical, with no eccentricity between the center of mass and the center of rigidity. This is approximate only, because the actual physical situation clearly indicates eccentricities, especially below the second-floor level.

At each story, the concrete floor slab was assumed to act as a rigid horizontal diaphragm. This is thought to be a reasonable assumption for two reasons: the relationship between the width and depth dimension of the diaphragm, and the existence of only a few large openings in the floor slabs. The largest diaphragm width-to-depth ratio is approximately 3.7.

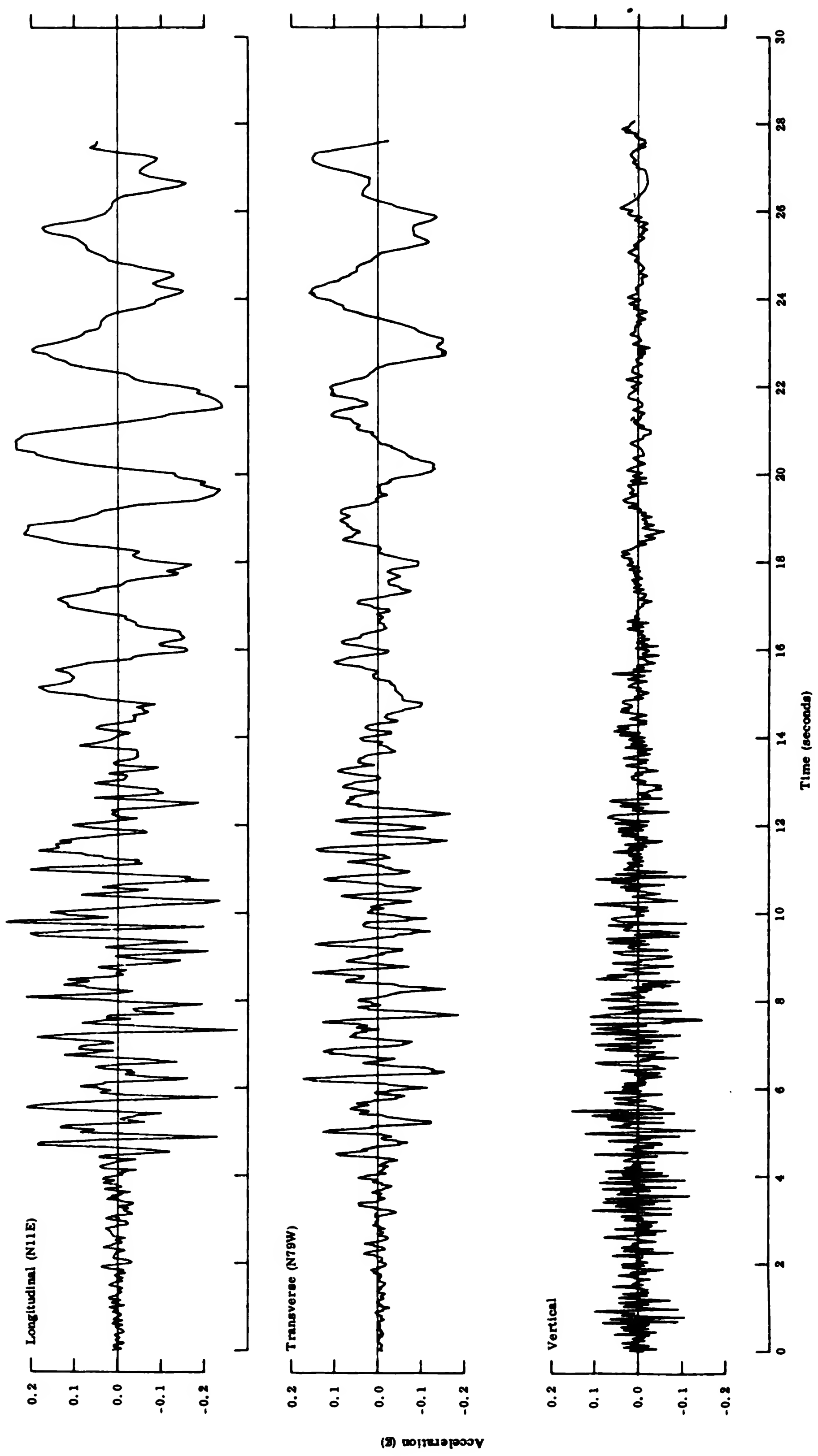


Figure 27.—Bank of California. Recorded acceleration at the roof.

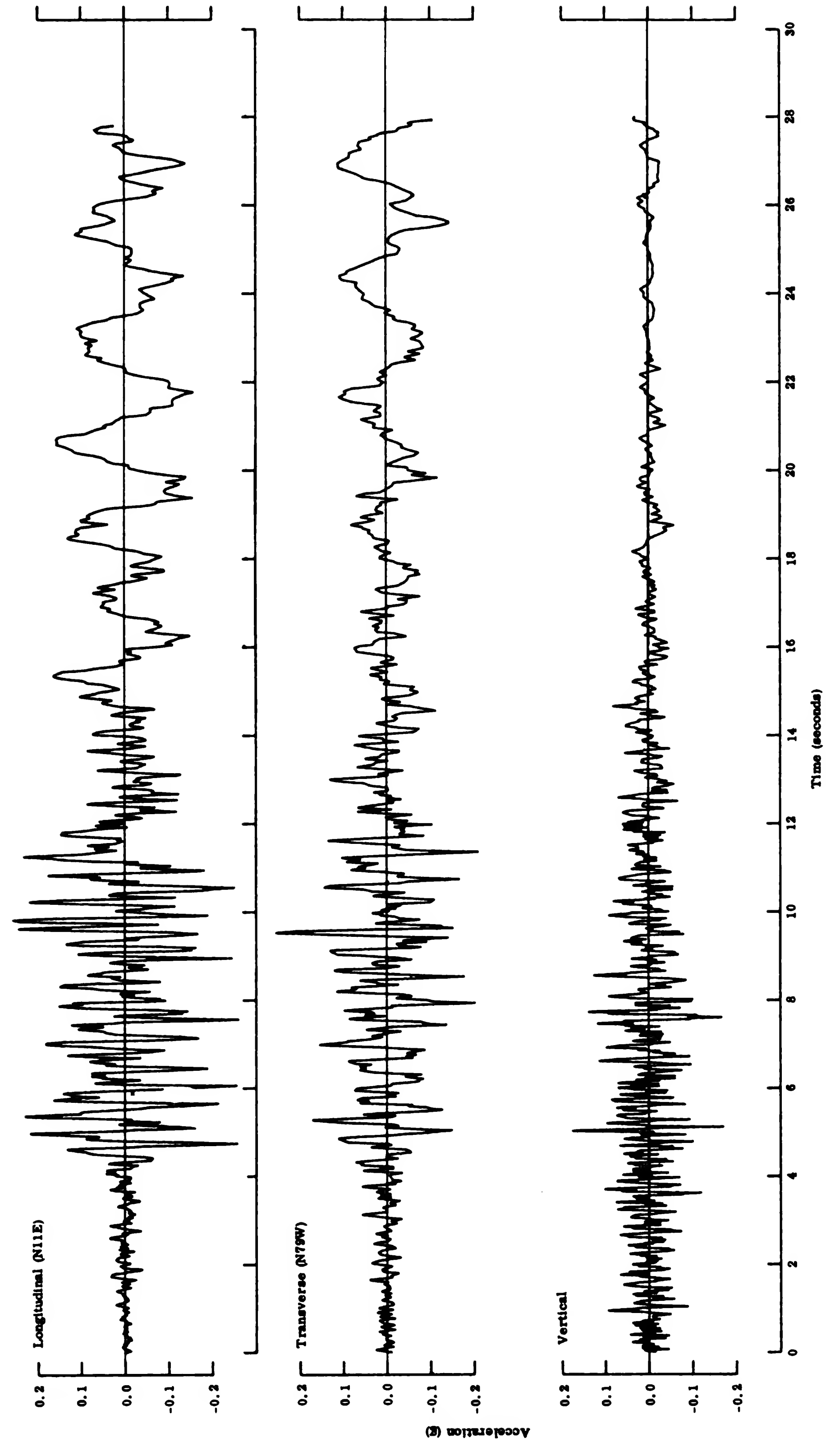


Figure 28.—Bank of California. Recorded acceleration at the seventh floor.

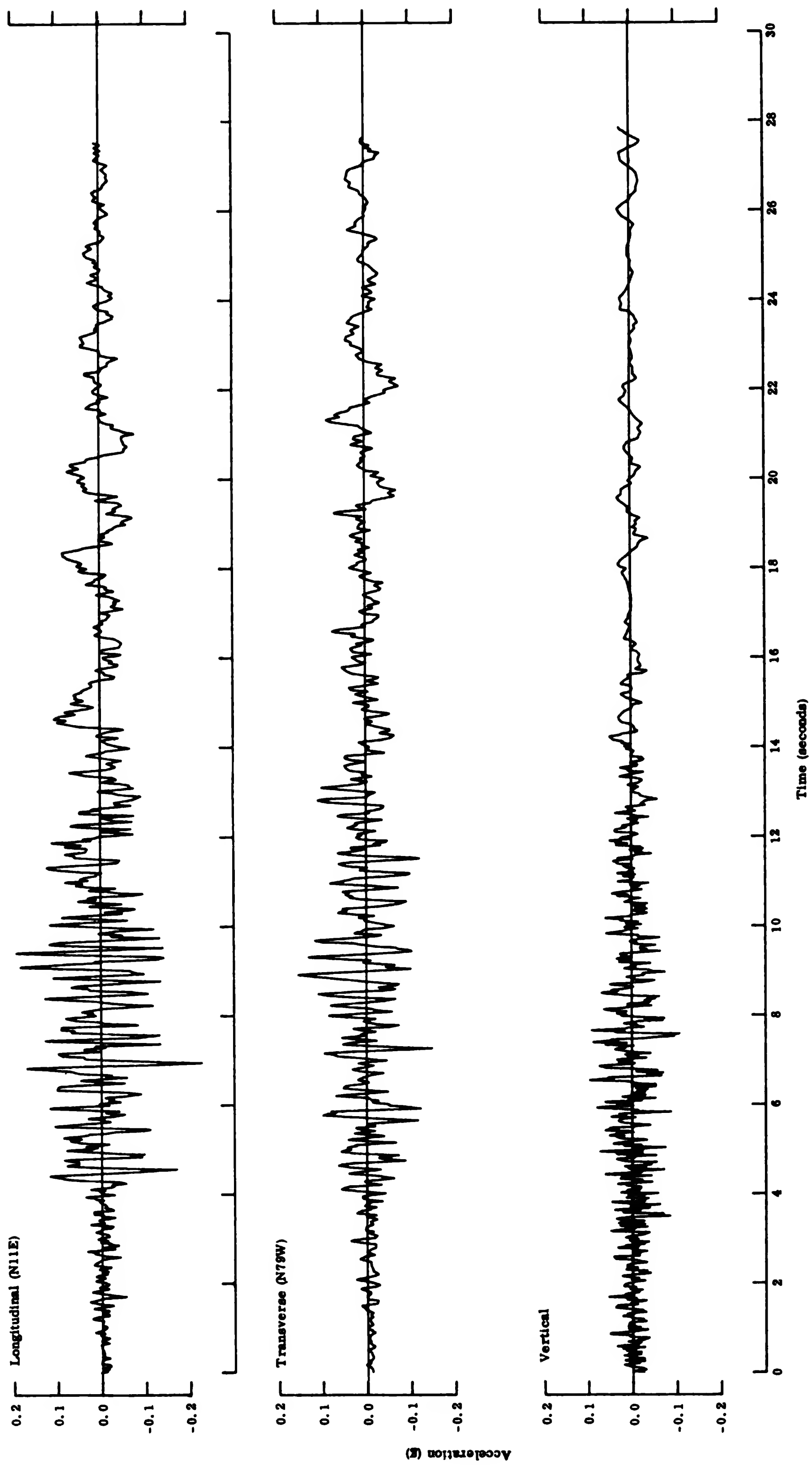


Figure 29.—Bank of California. Recorded acceleration at the ground floor.

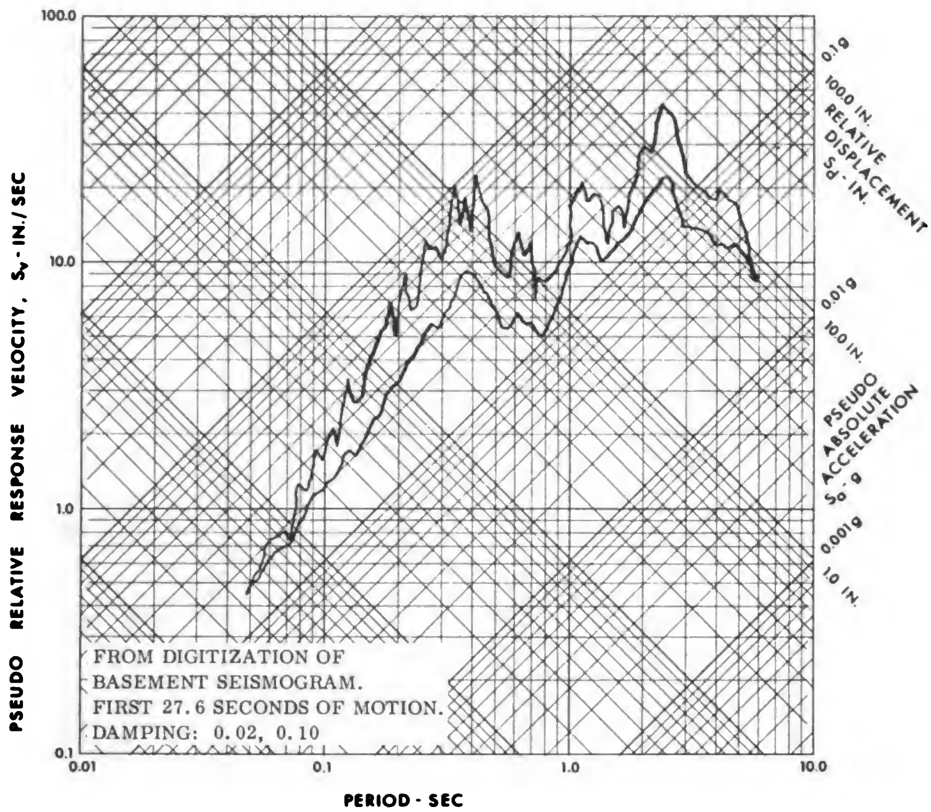


Figure 30.—Bank of California. Transverse response spectra.

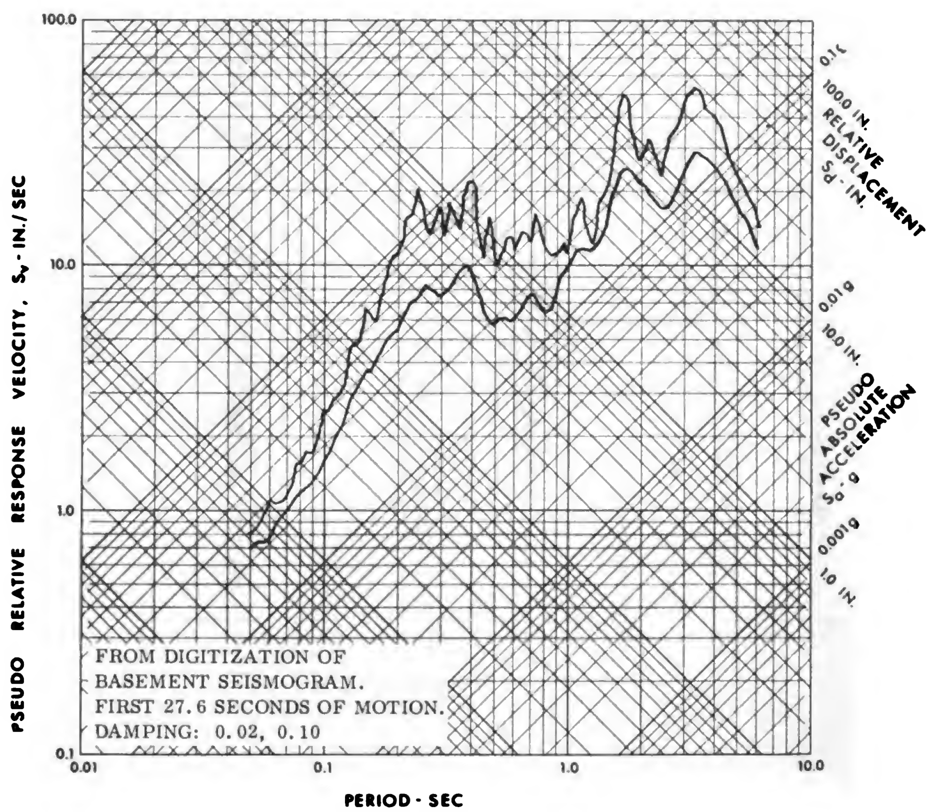


Figure 31.—Bank of California. Longitudinal response spectra.

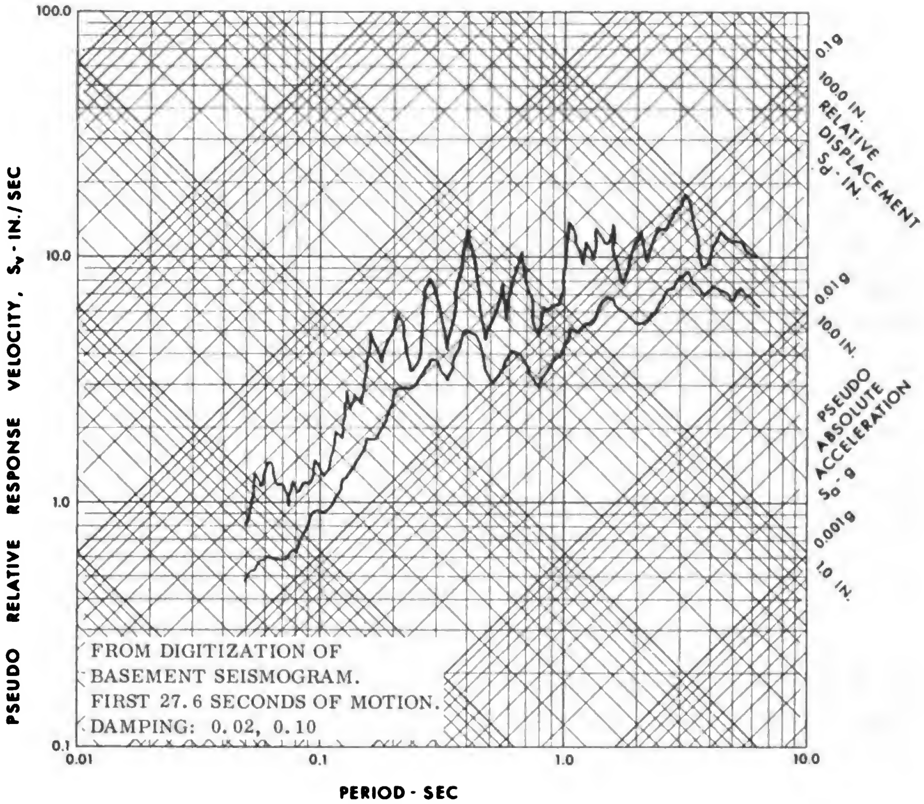


Figure 32.—Bank of California. Vertical response spectra.

Several models were used to investigate the response of the building. These are briefly summarized in table 4. Figures 33 and 34 show frames associated with each model. The number of models required was largely because of the inelastic nature of the building's response. As will be discussed more fully later, no single model was developed that would closely approximate recorded response throughout the entire earthquake excursion.

Initially, models were developed for each direction based upon estimated elastic properties. It was assumed that all structural elements participated in resisting lateral forces. These models were designated TA and LA.

Later models, TB and LB1, were developed to consider only elements specifically designed to resist lateral forces. Models TC2, LB2, and LB3 were among the models developed in an attempt to reconcile recorded and computed response. Table 4 summarizes the characteristics of the various models.

Damping values in table 4 seemed to give best correlation with recorded response. For each model, the same damping value was used with all modes.

Table 4.—Mathematical models used in the analysis

Building direction	Model	Fundamental period	Lateral force-resisting system	Purpose	Earthquake time interval	Number of modes	Applied viscous damping
		Seconds			*Seconds*		*Percent*
Transverse.......	TA	1.33	Two exterior and all interior frames.	Mode shapes and periods..................................			
Do.........	TB	1.60	Includes only frames designed to resist seismic loads.	Mode shapes, periods, and dynamic analysis.	0–28	6	5
Do.........	TC2	2.50	.....do....................	Dynamic analysis............	0–28	6	10
Longitudinal.....	LA	0.85	Two exterior and one interior frame.	Mode shapes and periods..................................			
Do.........	LB1	.93	Includes only frames designed to resist seismic loads.	....do..			
Do.........	LB2	1.50	.....do.....................	Dynamic analysis............	0–28	3	5
Do.........	LB3	1.80	.....do........................	.do.....................	0–28	3	10

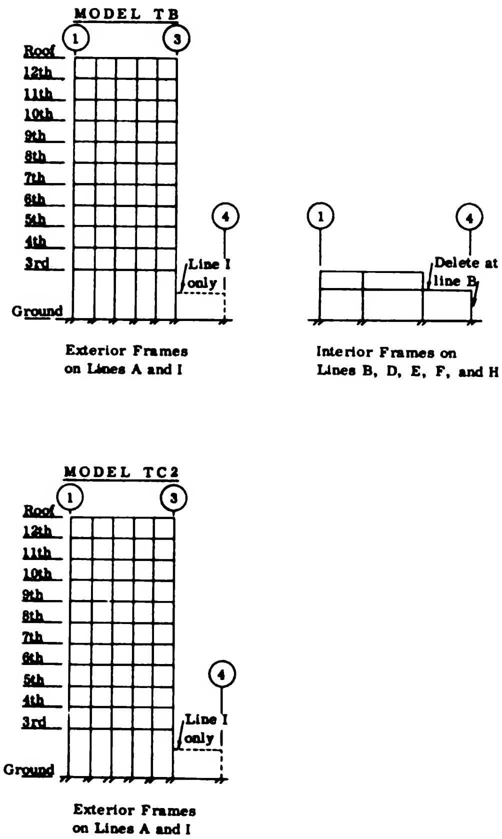


Figure 33.—Bank of California. Typical transverse frames.

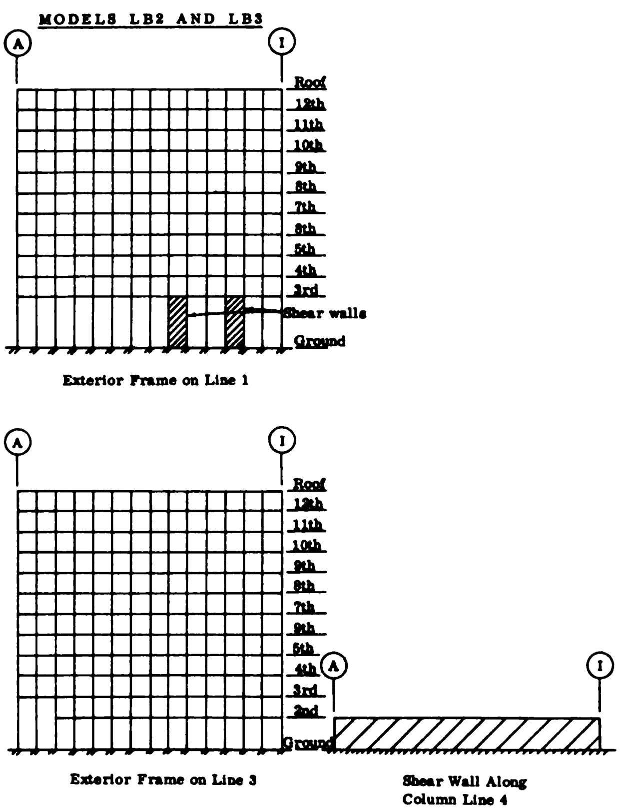


Figure 34.—Bank of California. Typical longitudinal frames.

RESULTS OF ANALYSIS

As described in the previous section, several mathematical models were developed for each principal direction. Initially, these models were developed from a careful assessment of the expected actual stiffness characteristics and mass distribution of the structure.

Subsequent analysis showed that the building described in table 5 generally responded with a lower frequency response than either set of initial mathematical models (TA and LA or TB and LB1). Consequently, several additional mathematical models had to be developed to provide correlation for different time intervals of the earthquake motion. The models used to determine the results presented herein are TB and TC2 for the transverse direction, and LB2 and LB3 for the longitudinal direction.

Table 5.—Apparent building characteristics during earthquake

Building direction	Earthquake time interval	Fundamental period	Equivalent viscous damping
	Seconds	*Seconds*	*Percent*
Transverse........	0–5	1.33	5
	5–15	1.65	5
	15–21	2.0	5
	21–28	2.5	10
Longitudinal......	0–3	1.15	5
	3–15	1.5	5
	15–21	1.8	10
	21–28	2.5	10

Mode Shapes and Periods of Vibration

Mode shapes and periods of vibration for models TB and LB1 were calculated for the first three translational modes in the longitudinal direction and the first six modes in the transverse direction. The first three mode shapes for both directions are shown in figure 35.

Computed Floor Accelerations

Actual earthquake accelerations were recorded in the three principal directions by strong-motion instruments located at the roof, seventh floor, and ground floor. Plots of the digitized records of the acceleration time histories are shown in figures 27, 28, and 29. As a principal part of this seismic study, correlation studies of recorded versus computed floor accelerations were made. Results of these studies, shown as recorded and computed accelerations, are

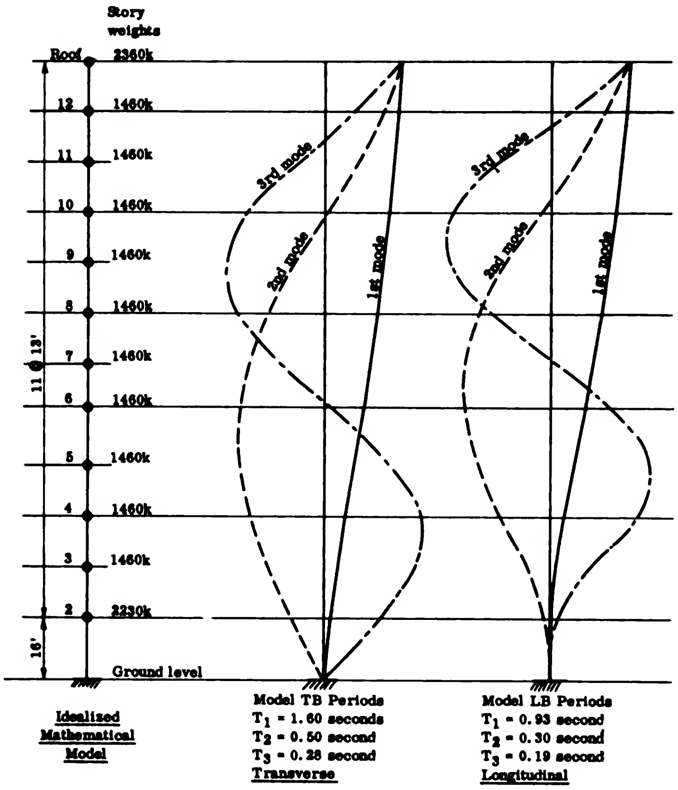


Figure 35.—Bank of California. Calculated periods and mode shapes.

presented in figures 36 and 37 for the transverse and longitudinal directions at both the roof and seventh-floor levels. Table 6 provides a comparison of recorded and computed peak floor accelerations.

Maximum Building Displacements

Envelopes of both maximum total displacement and maximum interstory displacement are shown in figures 38a and 39a. These represent the peak displacement of each story with respect to the base and the peak floor-to-floor differential displacement. Not all of the maximums occurred simultaneously. In the transverse direction, maximums occurred between 22.5 and 23.4 seconds after the start of motion, while in the longitudinal direction maximums occurred between 19.2 and 21.5 seconds after the start of motion. Different models for each direction showed maximums at nearly identical times.

Maximum Story Forces

Figures 38b and 39b plot maximum horizontal story forces from the dynamic analysis and corresponding code values. For each level, dynamic story forces were determined as the product of the story mass times the maximum absolute story acceleration.

Maximum Story Shears

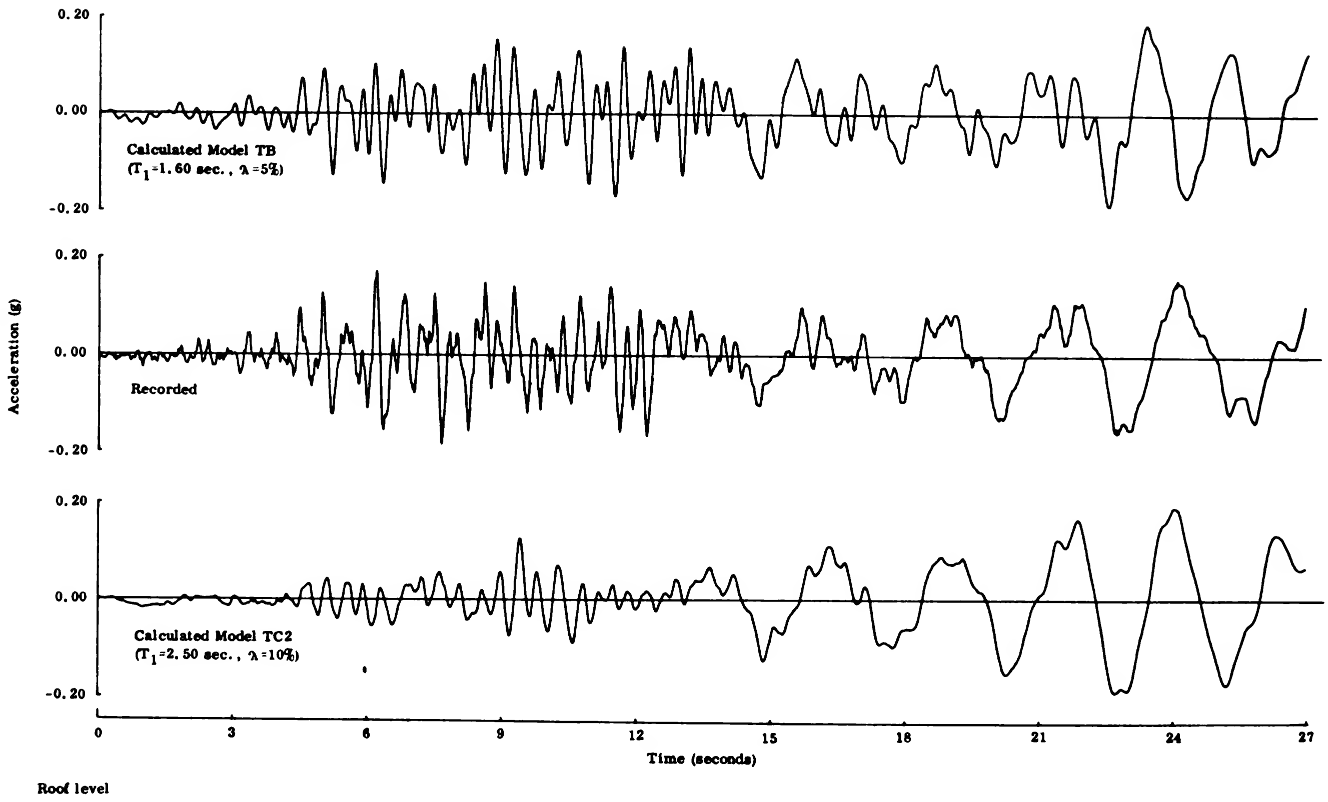
Maximum story shears were calculated as part of the dynamic analysis. These are indicated in figures 38c and 39c, along with corresponding code values. Code design shears have been computed for the requirements of the 1970 UBC (reference 4). The UBC story shears are based on the numerical coefficients and periods given in table 7.

The story shears determined from the dynamic analysis, indicated in figures 38c and 39c as idealized smooth curves, are peak values. They were calculated to have occurred at some time during the time-history response. These peak values did not necessarily occur simultaneously.

Figures 38c and 39c represent only an envelope of maximums. The story shears indicated are due to the contribution of either the first three (longitudinal)

Table 6.—Maximum accelerations and displacements at roof and 7th floor

Direction of response	Maximum parameter	Roof		7th floor	
		Model			
		TB	TC2	TB	TC2
Transverse	Computed acceleration (g)	0.20	0.19	0.21	0.14
	Recorded acceleration (g)	0.19		0.26	
	Computed displacement (in.)	4.3	11.0	2.5	7.5
	Computed story drift (in.)	0.17	0.31	0.47	0.86
		Model			
		LB2	LB3	LB2	LB3
Longitudinal	Computed acceleration (g)	0.27	0.27	0.18	0.17
	Recorded acceleration (g)	0.28		0.26	
	Computed displacement (in.)	5.5	8.6	3.0	4.7
	Computed story drift (in.)	0.20	0.31	0.65	1.02



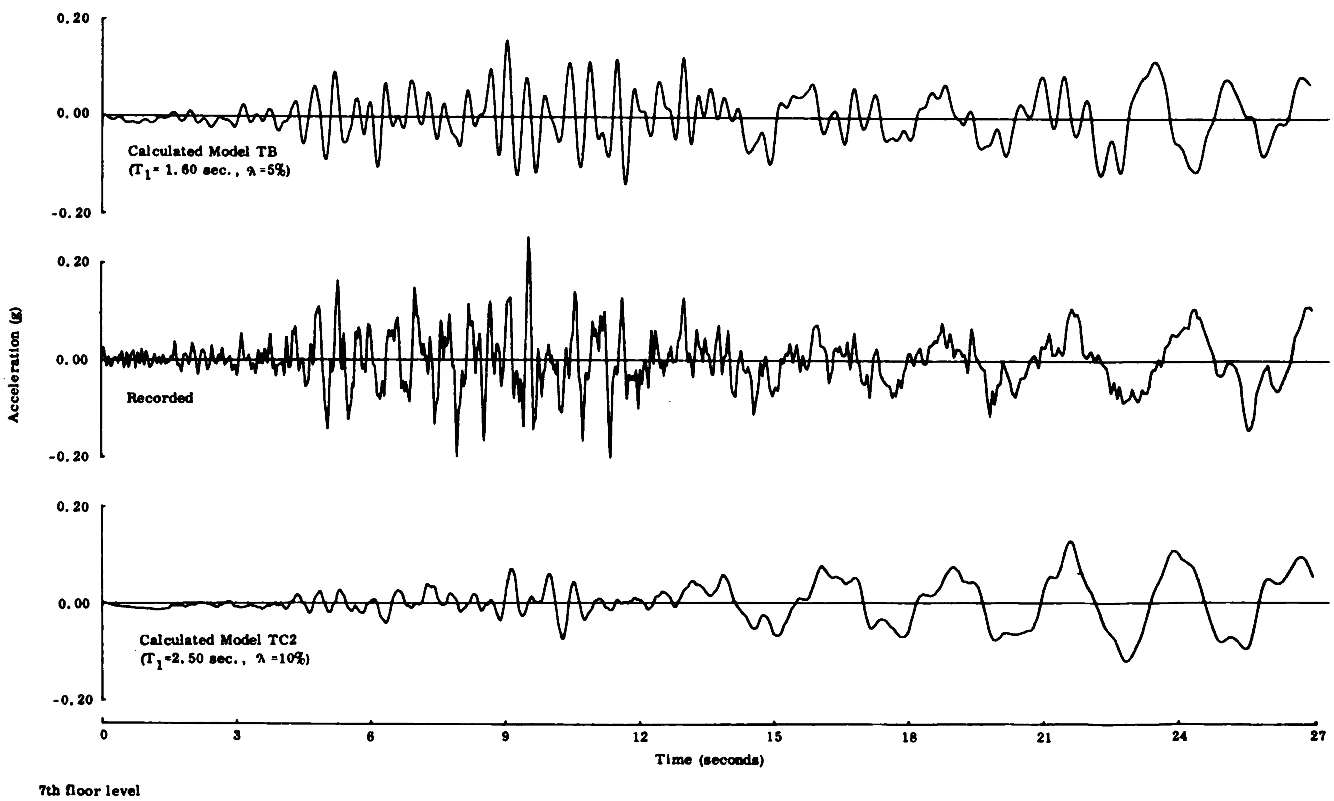


Figure 36.—Bank of California. Transverse calculated and recorded accelerations.

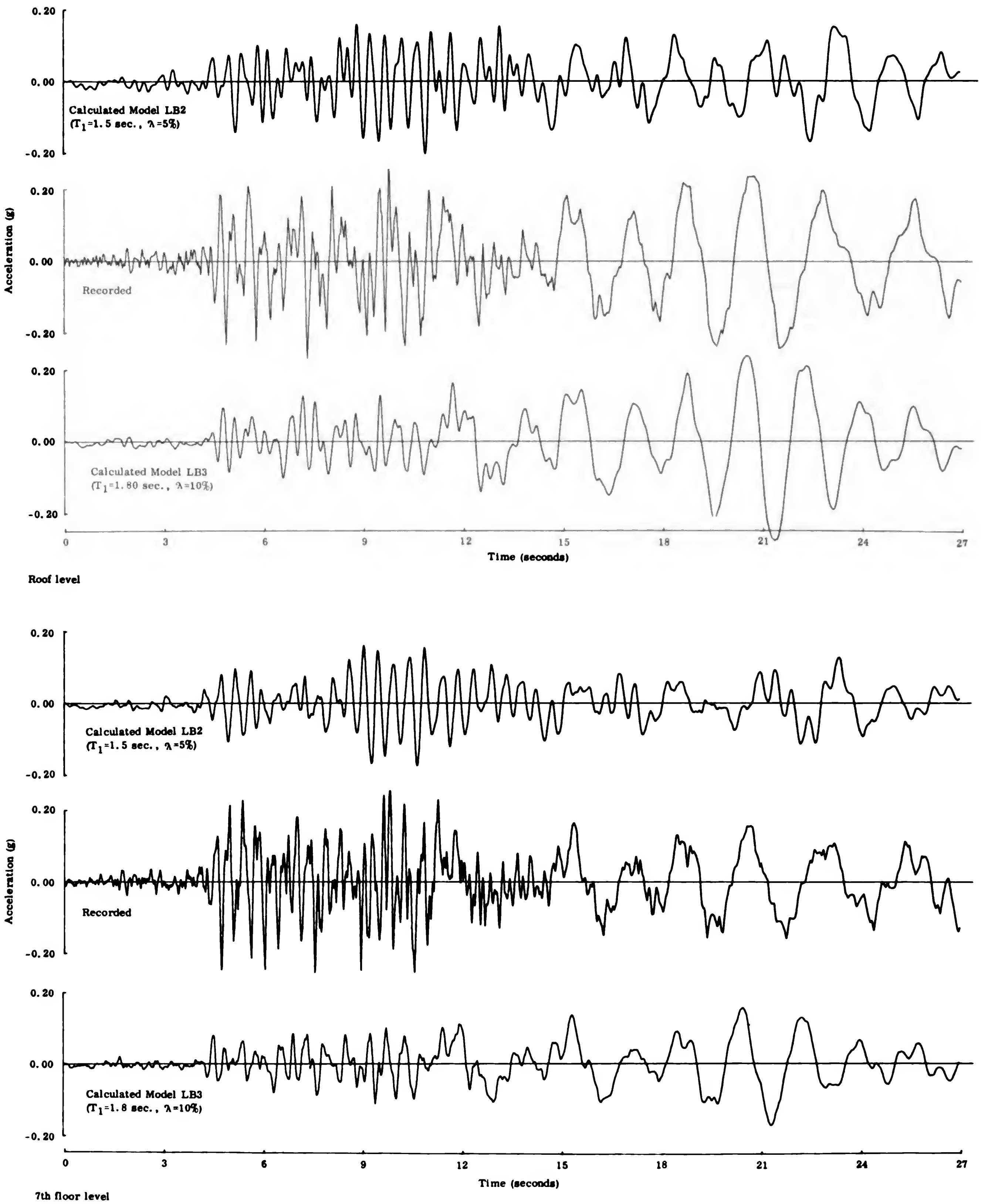


Figure 37.—Bank of California. Longitudinal calculated and recorded accelerations.

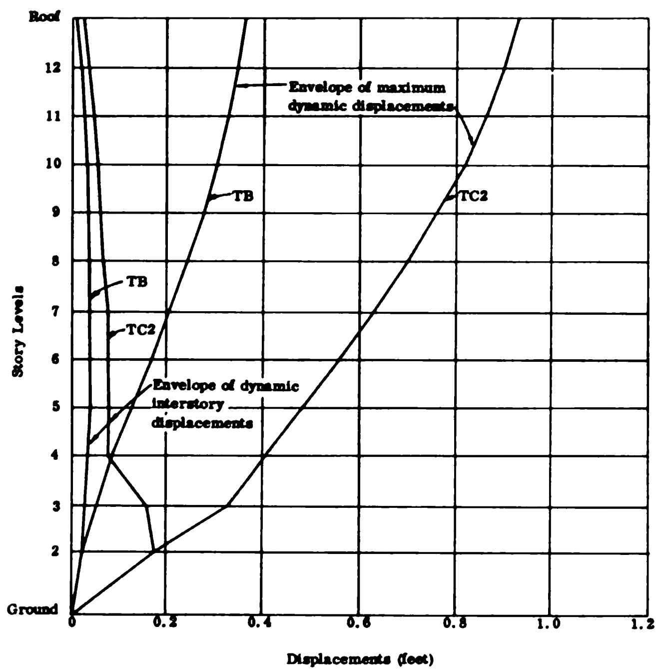


Figure 38a. TOTAL BUILDING DISPLACEMENTS AND INTERSTORY DISPLACEMENTS

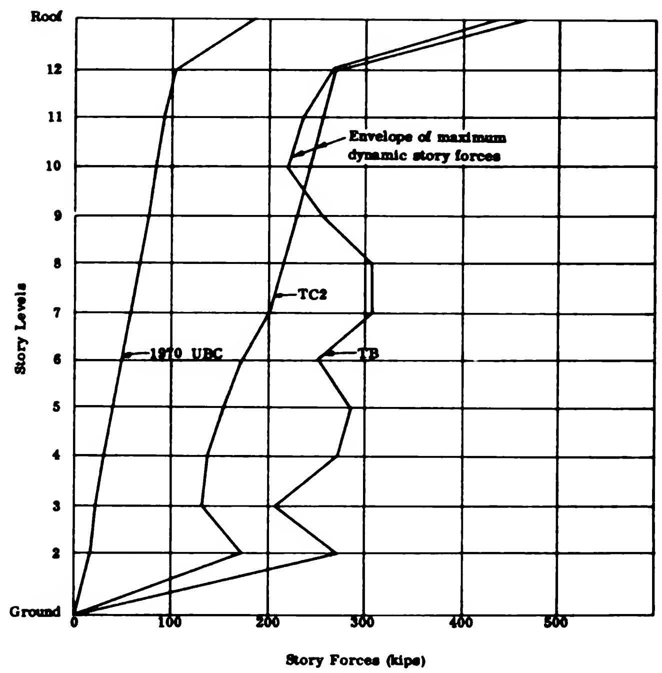


Figure 38b. MAXIMUM STORY FORCES

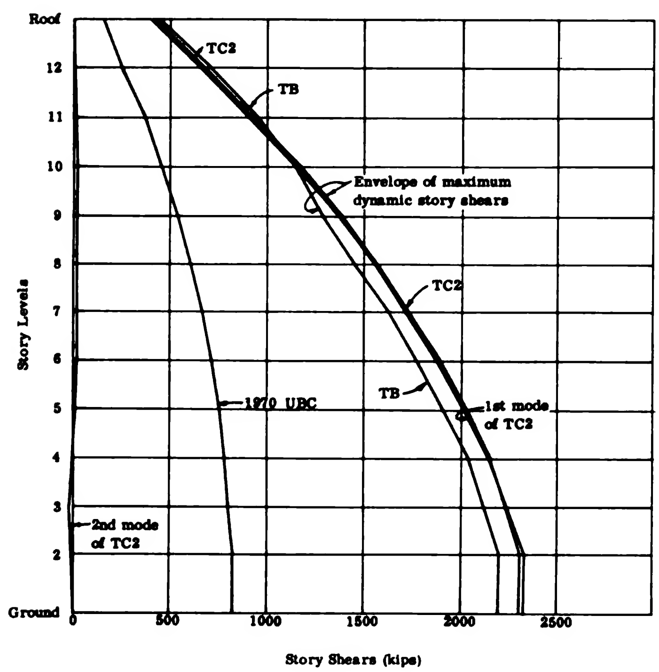


Figure 38c. MAXIMUM STORY SHEARS

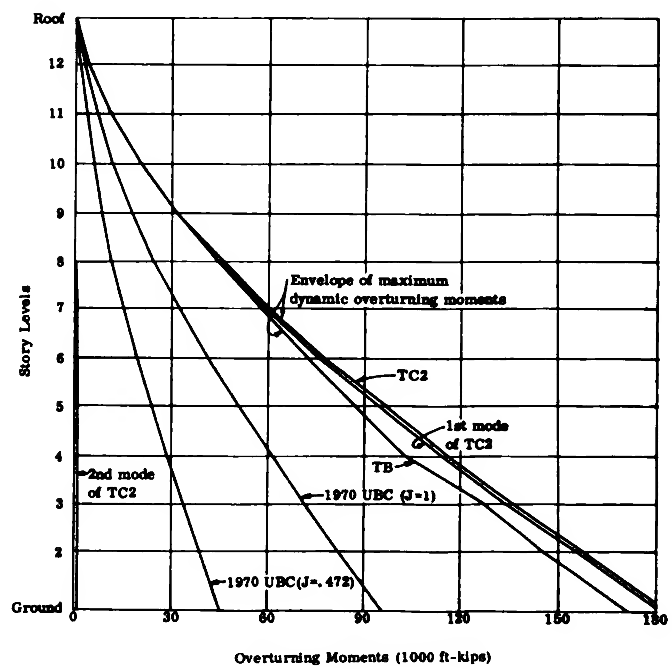


Figure 38d. MAXIMUM OVERTURNING MOMENTS

Figure 38.—Bank of California. Dynamic response and design code values for transverse (east-west) direction.

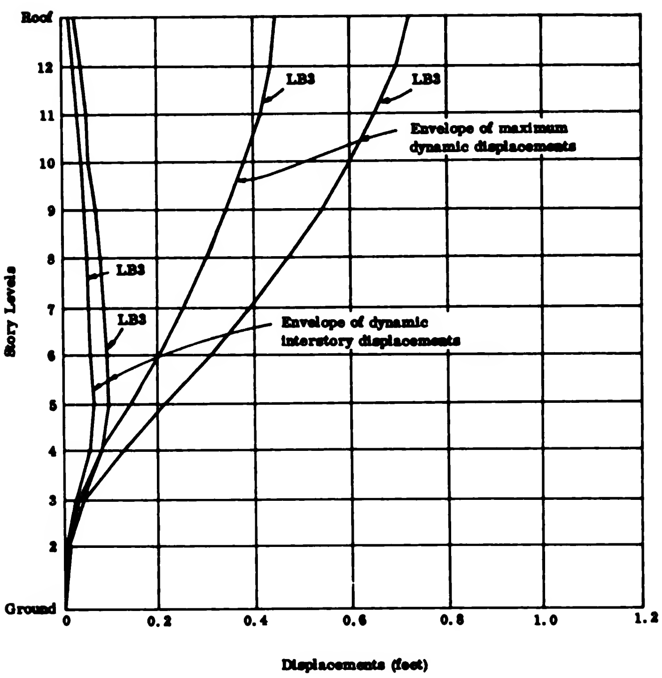


Figure 39a. TOTAL BUILDING DISPLACEMENTS AND INTERSTORY DISPLACEMENTS

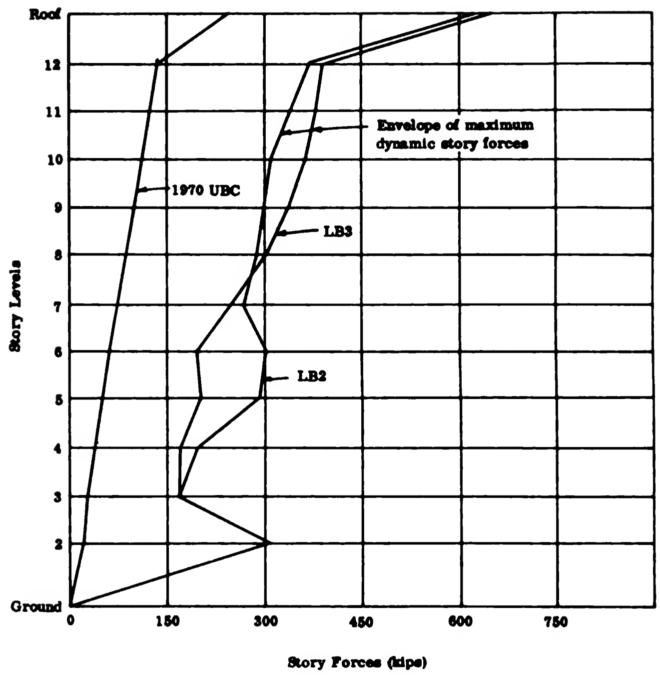


Figure 39b. MAXIMUM STORY FORCES

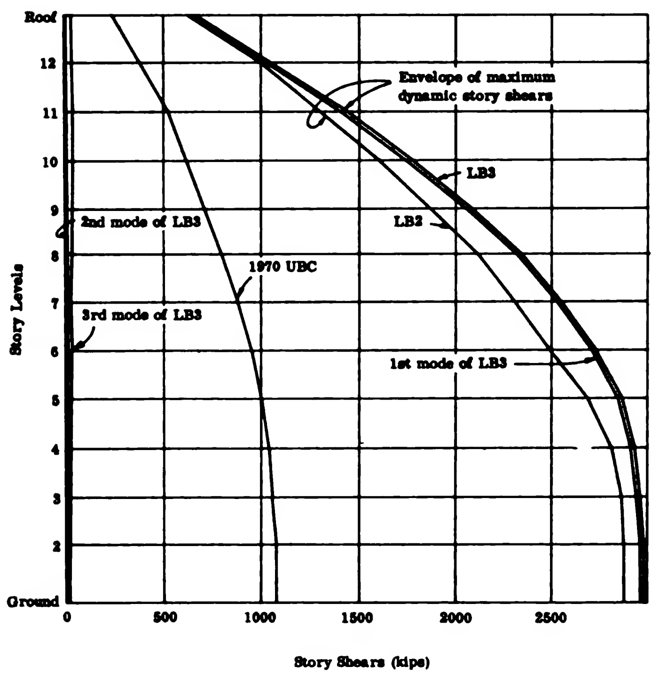


Figure 39c. MAXIMUM STORY SHEARS

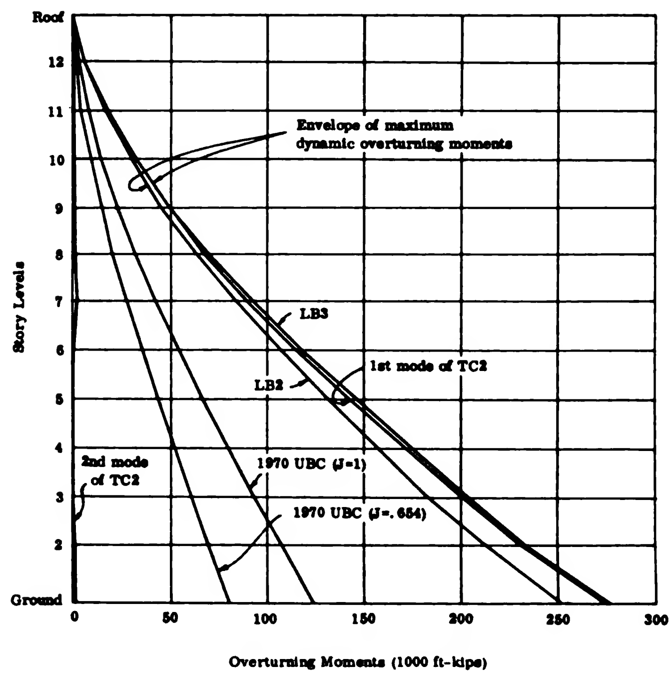


Figure 39d. MAXIMUM OVERTURNING MOMENTS

Figure 39.—Bank of California. Dynamic response and design code values for longitudinal (north-south) direction.

Table 7.—UBC seismic design parameters

Code	Building direction	Z	K	C	Base shear V = ZKCW	Period (T)	J
						Seconds	
1970 UBC..	Transverse.	1.0	1.0	0.044	0.044W	1.09	0.47
1970 UBC..	Longitudinal.	1.0	1.0	.057	.057W	.67	.65

or the first six (transverse) modes of vibration. The first-mode contribution accounts for almost all of the total response. As can be seen in figure 39c, the second and third modes contribute little to the maximum story shears.

Maximum Overturning Moments

Envelopes of maximum overturning moments were calculated in a manner similar to that used to determine the maximum story shears. Dynamic overturning moment values and those determined under 1970 UBC requirements are shown in figures 38d and 39d. As in the case of story shears, the first three or six modes of vibration were considered. The contributions of the first and second modes at the time of maximum response also are shown in figures 38d and 39d. First-mode response provides by far the largest contribution.

Overturning moments, calculated in the dynamic analysis under minimum code requirements of the 1970 UBC, and for the 1970 UBC with $J = 1.0$, have been shown in figures 38d and 39d. After publication of the 1970 UBC, the J factor for determining the base overturning moment received considerable scrutiny. Table 7 indicates that the 1970 UBC would require minimum J values of 0.47 and 0.65 for the transverse and longitudinal directions. However, subsequent amendments to the 1970 UBC have, in effect, increased the minimum J value to 1.0. For this reason, overturning moments determined with a value of $J = 1.0$ also have been included in figures 38d and 39d. These provide a comparison with the prescribed 1970 UBC minimums.

Loads on Key Girders

Earthquake loads on selected key girders of the lateral force-resisting frames were investigated by calculating combined seismic and estimated actual vertical loads. Then, these values were compared to the estimated ultimate load capacity of each member.

Specifically, comparisons were made by calculating ratios of the controlling combinations of vertical and seismic moments to estimated ultimate moment capacities. These ratios were determined from models TB and LB2 since results from models TC2 and LB3 are not significantly different. Table 8 indicates these as M/M_u. Ultimate moment capacities (M_u) were computed on the basis of the recommendations of reference 3. In these calculations, a capacity reduction factor of $\phi = 1.0$, instead of the usual 0.9 design value, was used.

Table 8.—Summary of girder M/M_u

Frame	Floor	Girder size B x D	M/M_u [1] Exterior joint	M/M_u [1] Typical interior joint
		Inches		
Transverse frame on line A (model TB).	Roof...	12 by 45⅞....	1.0	1.9
	10th...	12 by 40......	0.9	1.6
	7th....	12 by 40......	1.0	1.6
	5th....	12 by 40......	1.2	1.7
	3d.....	12 by 48......	1.2	1.2
Longitudinal frame on line 1 (model LB2).	Roof...	10 by 45⅞....	.5	0.7
	10th...	10 by 40......	.5	1.1
	7th....	10 by 40......	.6	1.1
	5th....	10 by 40......	.7	1.3
	3d.....	10 by 48......	.5	1.0

[1] Values of M/M_u are based either on top bars or bottom bars, whichever resulted in the greater value of M/M_u. M/M_u values greater than 1 indicate fictitious moments and should be interpreted only as general indicators that yield (or incipient yield) conditions existed in the reinforcement.

Results of the girder investigation revealed that reinforcement in some members yielded substantially. In table 8, these members would have M/M_u ratios much greater than 1, but the M_u term more closely associates with yield conditions in the girder reinforcement than with any true ultimate or failure condition. Generally, girders were under-reinforced. This indicated that the cross sectional area of available steel reinforcement, rather than crushing the concrete, limited ultimate moment capacity.

Because of the limitations of the modeling techniques and other analytical procedures used in this study, the results presented in table 8 should be viewed only as approximate indices of whether or not yield or failure conditions were exceeded.

Shear loads in girders, due to combined vertical and seismic forces, also were checked, but ultimate shear capacities, determined under the recommendations of reference 3, were not exceeded in general.

Loads on Key Columns

The results of a study of combined vertical and seismic loads on selected key columns are summarized briefly in table 9. First, ratios of combined vertical and seismic moments, (M), to estimated ultimate moment capacity, (M_u), were calculated for each principal axis of bending of key members. Then, values of summation of M/M_u (for biaxial bending) were determined.

Values greater than 1.0 would indicate yield or failure conditions may have been exceeded. It should be noted that the values indicated in table 9 are highly dependent on the assumptions implicit in the analysis, and results presented should be interpreted only as general indicators of how the columns responded to the earthquake. Results are based on models TB and LB2.

Estimates of ultimate moment capacities were based on reference 3. Axial forces were considered. A capacity reduction factor of $\phi = 1.0$ was used for the rectangular tied columns. Since the calculations were for analysis rather than design considerations, the more usual ϕ-factor of 0.70 was not used.

The term, P_{uo}, in table 9 represents the ultimate axial load capacity of the column without bending moment present. Values of P/P_{uo}, where P is the applied load, are given to provide an approximate index of axial load effects. The summations of M/M_u values (for biaxial bending) are also approximate indices of a more complicated interaction relationship.

Shear forces in columns were reviewed. Maximum values of nominal shear stress under maximum story shears ranged to about 300 psi. Under this level of shear stress, column ties probably participated in carrying shear forces. No evidence of distress attributable to shear forces, however, was observed, and column shears consequently were not investigated extensively. The matter, however, does need special investigation, particularly corner columns and columns subjected to net uplift forces (reference 9).

DISCUSSION AND INTERPRETATION OF RESULTS

Comparison of Calculated Versus Code Forces

In the previous paragraphs, results of the dynamic analysis and subsequent comparisons of those results with code design values were presented. The results of the dynamic analysis generally showed that the level of code seismic forces was substantially less than what the structure was required to resist. Examination of the computed member forces revealed that, in many cases, loads due to combined vertical and seismic forces caused stresses in reinforcement that exceeded yield levels.

Calculated linear response proved difficult to reconcile with recorded motion, especially over the en-

Table 9.—Summary of column interaction

Column	Floor	Column size	Transverse frame (Model TB) [1]		Longitudinal frame (Model LB2) [1]	
			$\frac{P}{P_{uo}}$	$\frac{M_x}{M_{uw}} + \frac{M_y}{M_{uy}}$	$\frac{P}{P_{uo}}$	$\frac{M_x}{M_{ux}} + \frac{M_y}{M_{uy}}$
		Inches				
A1 (typical corner column)	Roof	(2)	0.2	0.9	0.14	0.5
	10th	(2)	.1	(3)	.03	.7
	7th	(2)	.8	(3)	.2	1.1
	5th	(2)	1.5	(3)	.6	(3)
	3d	(2)	2.2	(3)	.1	(3)
A2 (typical interior column in transverse frame).	Roof	18 by 36	.05	.9		
	10th	18 by 36	.08	1.9		
	7th	18 by 36	.09	1.4		
	5th	18 by 36	.10	1.5		
	3d	18 by 36	.11	.7		
D1 (typical interior column in longitudinal frame).	Roof	18 by 36			.02	.6
	10th	18 by 36			.05	1.0
	7th	18 by 36			.08	1.2
	5th	18 by 36			.09	1.3
	3d	18 by 36			.11	.3

[1] Values of M/M_u greater than 1 are fictitious and indicate only yield conditions in column reinforcement.
[2] See figure 5.
[3] Tension controls and corresponding P/P_{uo} values greater than 1 are fictitious and should be interpreted as indicating only yielding of reinforcement.

tire 28-second record. This is generally attributable to the fact that the structure changed stiffness and damping characteristics substantially during the earthquake. The large member forces as indicated by M/M_u ratios, the reduction of building stiffness as indicated by the lengthening periodicity of the recorded motion, and the numerous cases of cracking and spalling of concrete elements clearly evidence consistent and substantial nonlinear behavior.

The models used for each direction, despite their different characteristics, gave story forces, story shears, and overturning moments of approximately the same values as shown in figures 38 and 39. However, because of the substantial evidence of nonlinear behavior, all calculated response parameters should be viewed only as approximate indications of a building's true response.

Modal Analysis Procedures

To test the appropriateness of the models, an attempt was made to verify both mode shapes and periods. Unfortunately, the lack of sufficient data and the inelastic structural behavior prevented confirmation of calculated mode shapes. Records of motion were obtained for only the roof and seventh-floor levels. These would provide information for only two points of a building's mode shape.

Difficulties also arose when comparing the calculated and recorded building periods. This comparison indicated the limitations of the linear-elastic mathematical models, because no single model showed accurate correlation with measured acceleration responses for the entire length of recorded motion (table 5).

The low level of response during the first 5 seconds of earthquake motion limited accurate estimates of the fundamental periods for this time increment. It is believed, though, that the structure did remain elastic during the first 5 seconds of motion.

After 5 seconds, however, periods lengthened, apparently because of two factors. First, nonstructural elements, such as interior partitions and exterior enclosure walls, added stiffness to the structure during the relatively low-amplitude, initial-motion period of the earthquake. As amplitudes of motion increased, the partitions cracked or separated from the structure, increasing the predominant periods of response.

Second, yielding occurred in columns and girders of both the seismic and nonseismic frames, causing further lengthening of periods. A member forces investigation indicated that a substantial number of both girders and columns were loaded beyond the yield levels of their reinforcement. From the recorded periods, it appears that yielding in the transverse direction began in the nonseismic frames after about 5 seconds of motion and in the seismic frames after about 15 seconds. In the longitudinal direction, yielding began in both nonseismic and seismic framings after about 4 seconds.

The modal analysis procedures also were hindered by the structure's apparent change in damping characteristics during the earthquake. Higher viscous damping values gave improved correlation for the later part of the earthquake record (table 5). This correlated with the observed cracking and working of concrete and the calculated yielding of reinforcement that would have been expected to increase damping. Hysteretic effects could only be modeled very approximately with higher viscous damping values.

Comparisons of Recorded and Computed Responses

Comparisons of recorded and computed responses were made by comparing acceleration time histories for the roof and seventh-floor levels. Table 6 lists peak recorded accelerations and computed values of peak acceleration, maximum total displacement, and maximum interstory drift for the various models. The general shape of the computed time history of the acceleration response in both directions and at both the roof and seventh floor did not correlate well with the recorded values (figs. 36 and 37). Only parts of the computed motion correlated because of the general nonlinear response of the structure.

Figure 37 shows two computed and one recorded acceleration time-history plot for the longitudinal roof component. That part of the computed response for model LB3 between 13 and 21 seconds correlates reasonably in periodicity and amplitude with the recorded results. However, at approximately 21 seconds, the recorded accelerations reach a relatively broad peak of high amplitude. The recorded accelerations immediately following the 21-second mark show a pronounced lengthening of period. This can be attributed to the pulse at 21 seconds causing additional yielding of reinforcement, cracking, and spalling of concrete, thereby lengthening the building's predominant period of response from approximately 1.8 to roughly 2.5 seconds.

The first 13 seconds of computed response for model LB3 correlates poorly with recorded values. The recorded acceleration values show higher amplitudes and a higher frequency of response. This indicates that during the initial 13 seconds of the building's response, damping was less and stiffness greater than that modeled by model LB3. Other models exhibit similar difficulties.

Correlation With Damage Observations

Results of the dynamic analysis indicate that some structural damage was to be expected from the earthquake. This was verified by the field damage survey described earlier. Damage to structural elements consisted of cracking and spalling of concrete and localized yielding of reinforcement. Nonstructural damage to partitions, stairs, stairwells, mullions, building contents, mechanical equipment, and architectural concrete elements (nonstructural concrete members) also correlates with the level of computed response.

Yielding of the nonseismic frames occurred very early in the earthquake. These frames were designed only for vertical loads, but, initially, because of their inherent stiffness characteristics, tended to carry substantial seismic loads.

One of the weak points in these frames occurred at the exterior column-to-girder connection. Here the bottom reinforcement was anchored insufficiently to the column and spalling subsequently occurred.

Cracking of the floor slab around the columns at the sixth and eighth floors on both the west and the east sides also was caused by nonseismic frames carrying seismic forces. In this case, the frames transferred forces to the slab at the girder-column connection.

Yielding was calculated to have occurred also in the seismic frames, with both girders and columns loaded above the yield levels of their reinforcement. Considerable cracking and spalling at the juncture of girders and columns were observed on the east and west sides of the building. This correlates with the analysis, which showed that both the columns and girders in this area were loaded over their yield moment capacities, but this also relates to the construction joint detail (fig. 18) used above the fourth floor. Where this detail applies, yielding of the top reinforcement was unlikely.

If the structure had remained elastic, the lateral displacement of the roof in the transverse direction would be expected to be approximately 4.3 inches. Calculated maximum dynamic displacements at the roof for models corresponding to the apparent maximum recorded periods (TC2 and LB3) are 11.0 inches and 8.6 inches in the transverse and longitudinal directions. Because of the large magnitude of these displacements, interstory displacements also would be expected to be large, as indicated in table 6 and figures 38a and 39a. Calculated displacements for models TC2 and LB3 can be considered to correlate accurately with the damage illustrated in figures 24, 25, and 26.

Vertical Accelerations

Vertical accelerations recorded at the roof and seventh floor show significant amplification over that recorded at the ground-floor level. Vertical accelerations at all levels have lesser amplitudes than those recorded for either of the two horizontal components, but the vertical acceleration record has a greater content of high-frequency motion.

SUMMARY OF FINDINGS

From a review of the results of the dynamic analysis and from a study of observed earthquake damage, the following conclusions have been drawn:

1 The building resisted the earthquake largely by inelastic means, with local yielding of reinforcement and with cracking and localized spalling of column and girder concrete.

2 The structural response of the building was nonlinear and cannot be well described or modeled by linear-elastic dynamic analysis techniques.

3 Calculated earthquake forces were substantially greater than prescribed code minimums. Calculated story shears were greater than 2.5 times code values, and calculated overturning moments were about 2.0 times code values with $J = 1.0$.

4 Architectural damage was related to interstory displacements and consisted largely of cracking, racking, and working of partitions. Some mechanical equipment and building contents were damaged by shaking alone.

5 Structural damage did occur, but it was limited to cracking and minor spalling of concrete. Yielding of reinforcement in localized areas also occurred.

RECOMMENDATIONS

As a result of this study, several areas of possible change in seismic design practice and building instrumentation have been recommended. The following paragraphs present these.

Code Seismic Forces

One possible means of providing structures with greater earthquake resistance is to increase minimum code seismic force requirements. If the present equivalent static force method of determining design seismic forces is retained, then an increased base shear and overturning moment requirement may reflect more closely the actual loading conditions that may be imposed on the structure. For some structures, a more realistic approach might require the use of dynamic analysis procedures, using a hypothetical design earthquake based on the seismic exposure and soil conditions of the site.

Ductile Member Characteristics

To protect structures against seismic overloads, members of lateral force-resisting systems should be provided with ductile characteristics. This is the ability to undergo large inelastic deformations, including cyclic loads at stresses beyond yield levels, without significant loss of load-carrying capabilities. The use of members designed with ductile characteristics greatly enhances the seismic resistance of moment-resisting concrete frames.

Concrete aggregates should not be brittle. Regular weight and lightweight aggregate concrete should not be mixed at important column-girder connections.

Shear failure must be precluded by shear reinforcement adequate to resist the maximum shears associated with the formation of plastic hinges in either columns or girders. The concrete in the vicinity of the girder-column joint must be confined properly by hoop reinforcement to maintain integrity, while the reinforcement yields beyond the elastic limit. Hoop reinforcement also must be provided in the girder-column joint to resist the high diagonal compressive forces in the panel zone.

Inelastic Frame Behavior

In order to insure the integrity of columns, lateral force-resisting frames should be designed to initiate and confine inelastic behavior to girders. To preclude secondary failure due to actual or accidental eccentricities, columns and girders should be proportioned reasonably compactly. Columns should be designed to remain elastic at all times to carry dead and live loads without collapse.

Consideration also should be given to the possibility of girder reinforcement with yield strengths much greater than minimum specified values. Overstrength reinforcement will tend to increase girder moment capacities and possibly cause hazardous inelastic column behavior. Probable overrun in minimum specified yield strengths should be investigated. This would help to prevent the possibility of compressive or shear failure of the concrete when plastic hinging is achieved.

Building Eccentricities

Eccentricities between the center of mass and the center of rigidity should be avoided. Considering solely its resistance to earthquake forces, a structure should not have abrupt changes in centers of rigidities between adjacent floors, especially in the lower stories of high-rise buildings.

Analysis of centers of rigidity, by assuming elastic behavior, may not be appropriate for structures where inelastic behavior is possible during an earthquake. Under inelastic action, buildings with moment-resisting frames in combination with shear walls may exhibit different centers of rigidities than under elastic conditions, especially if the shear walls remain elastic.

Seismic resistance can be improved if the configuration of structural frames is typical throughout all stories and if abrupt changes in story stiffnesses, story height, bay width dimensions, member strengths, and types of materials can be avoided.

Seismic Frames

Girders and spandrels should frame into columns without offsets or otherwise avoidable eccentricities. Offset spandrels and spandrels framing into girders, rather than into columns along column centerlines, should be avoided.

Nonseismic Frames

Frames not considered part of the lateral force-resisting system of a building should be designed to carry seismic forces in accordance with their actual rigidities, even though other elements (seismic

frames) have been designed to carry all of the total seismic force.

Architectural Elements

Expansion joints, flashings, partitions, and stairwells should be designed for seismic movements. The amount of movement to be designed into these elements should be based on maximum possible interstory drifts, rather than on deflections computed for the present level of code seismic forces.

Strong-Motion Instrumentation

Strong-motion recording devices should be placed to provide a record of true transverse and longitudinal motion without any undue contributions from torsional effects (real or accidental). Vertical records should be taken near the center of a building at or near a column to avoid local, and possibly anomalous, effects from flexible floor elements such as thin slabs.

Future analytical studies of building behavior during earthquakes, especially verification of mode shapes, would be aided if a sufficient number of recording instruments were placed between the top and ground floors to furnish accurate data on the inflection points of higher modes.

Holiday Inn (29)

8244 Orion Avenue, Van Nuys

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JOHN A. BLUME & ASSOCIATES, ENGINEERS

San Francisco, Calif.

DESCRIPTION OF BUILDING

The Holiday Inn is a seven-story reinforced concrete structure, with typical plan dimensions of about 62 by 160 feet (fig. 1). Located just east of the San Diego Freeway at Roscoe Boulevard, the structure stands about 13 miles south of the epicenter of the San Fernando earthquake. This area lies in the center of the San Fernando Valley, midway between the Santa Monica Mountains to the south and the San Gabriel and Santa Susana Mountains to the north.

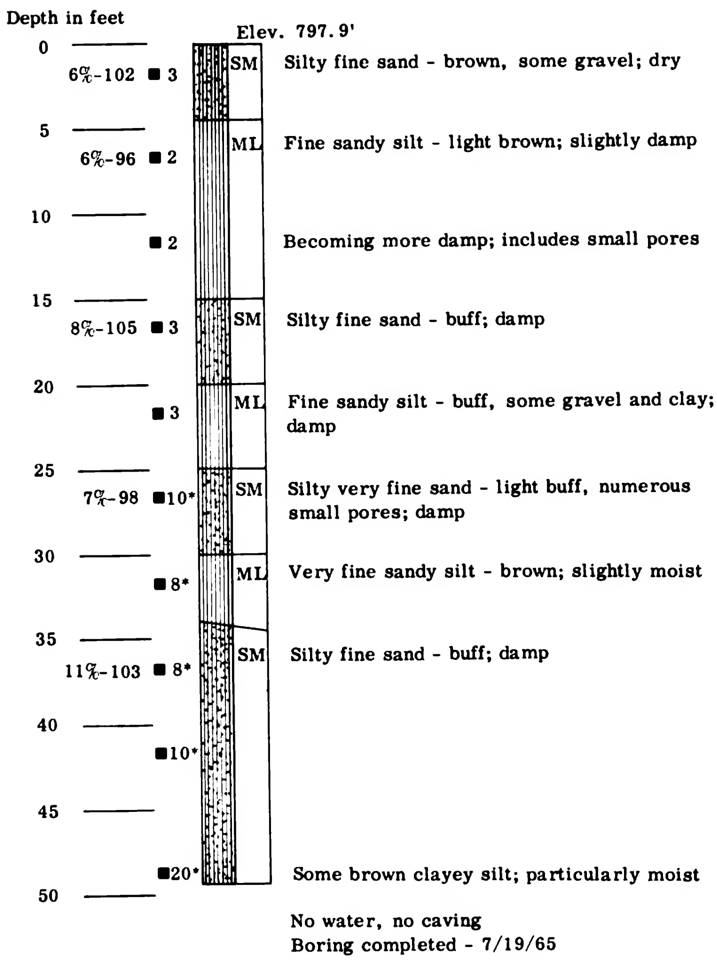
Geologic source data indicate that the site lies on recent alluvium. The typical soil boring log (fig. 2, reference 10) shows the underlying soil to be primarily fine sandy silts and silty fine sands.

The structure, which consists of roughly 63,000 square feet of floor area, was designed in 1965. Constructed in 1966 at a cost of approximately $1.3 million, it is essentially identical to the Holiday Inn at 1640 Marengo Street, discussed in Building Report 30.

The foundation system (fig. 3) consists of 38-inch-deep pile caps, supported by groups of two to four poured-in-place 24-inch-diameter reinforced concrete friction piles. These are centered under the main building columns. All pile caps are connected by a grid of tie beams and foundation beams. Each pile is roughly 40 feet long and has a design capacity of over 100 kips vertical load and up to 20 kips lateral load (reference 10).

The first floor is a slab on grade over about 2 feet of compacted fill. Except for two small areas at the ground floor, which are covered by one-story canopies, the plan configurations of each floor, including the roof, are the same (figs. 4, 5, and 6). The typical framing consists of columns spaced at 20-foot centers in the transverse direction and 19-foot centers in the longitudinal direction. Spandrel beams surround the perimeter of the structure. The floor system is a rein-

Figure 1.—Holiday Inn, Orion Avenue. Northwest elevation. John A. Blume & Associates photograph.



LEGEND
A-B■C
A Field moisture expressed as a percentage of the dry weight of soil.
B Dry density expressed in pounds per cubic foot.
C Blows per foot of penetration using a 2000 pound hammer dropping 12".
* - a 1200 pound hammer was used.
■ Depth at which undisturbed sample was extracted.

Figure 2.—Holiday Inn, Orion Avenue. Log of typical soil boring.

Table 1.—Properties of construction materials

Concrete (regular weight, 150 pcf[1] unit weight)

Location in structure	Minimum specified compressive strength (f'_c)	Modulus of elasticity (E)
	psi [2]	*psi* [2]
Columns, 1st to 2d floors	5,000	4.2×10^6
Columns, 2d to 3d floors	4,000	3.7×10^6
Beams and slabs, 2d floor	4,000	3.7×10^6
All other concrete, 3d floor to roof	3,000	3.3×10^6

Reinforcing steel

Location in structure	Grade	Minimum specified yield strength (f_y)	Modulus of elasticity (E)
		ksi [3]	*psi* [2]
Beams and slabs	Intermediate-grade deformed billet bars (ASTM A-15 and A-305).	40	29×10^6
Column bars	Deformed billet bars (ASTM A-432).	60	29×10^6

[1] Pounds per cubic foot.
[2] Pounds per square inch.
[3] Kips per square inch.

forced concrete flat slab, 10 inches thick at the second floor, 8½ inches thick at the third to seventh floors, and 8 inches thick at the roof (fig. 7). A penthouse with mechanical equipment covers approximately 10 percent of the roof area.

The structure is constructed of regular weight reinforced concrete. Table 1 gives the properties of the structural materials specified for the construction.

Interior partitions, in general, are gypsum wallboard on metal studs. Cement plaster, 1 inch thick, is used for exterior facing at each end of the building and at the stair and elevator bays on the long side of the building. Double 16-gauge metal studs support the cement plaster.

Some additional cement plaster walls are located on the south side of the building at the first floor. The north side of the building, along column line D, has four bays of brick masonry walls located between the ground and the second floor at the east end of the structure. Nominal 1-inch expansion joints separate these walls from the exterior columns. Nominal ½-inch expansion joints separate the walls from the underside of the second-floor spandrels. Although none of the wall elements described are designed as part of the lateral force-resisting system, they do contribute in varying degrees to the stiffness of the structure.

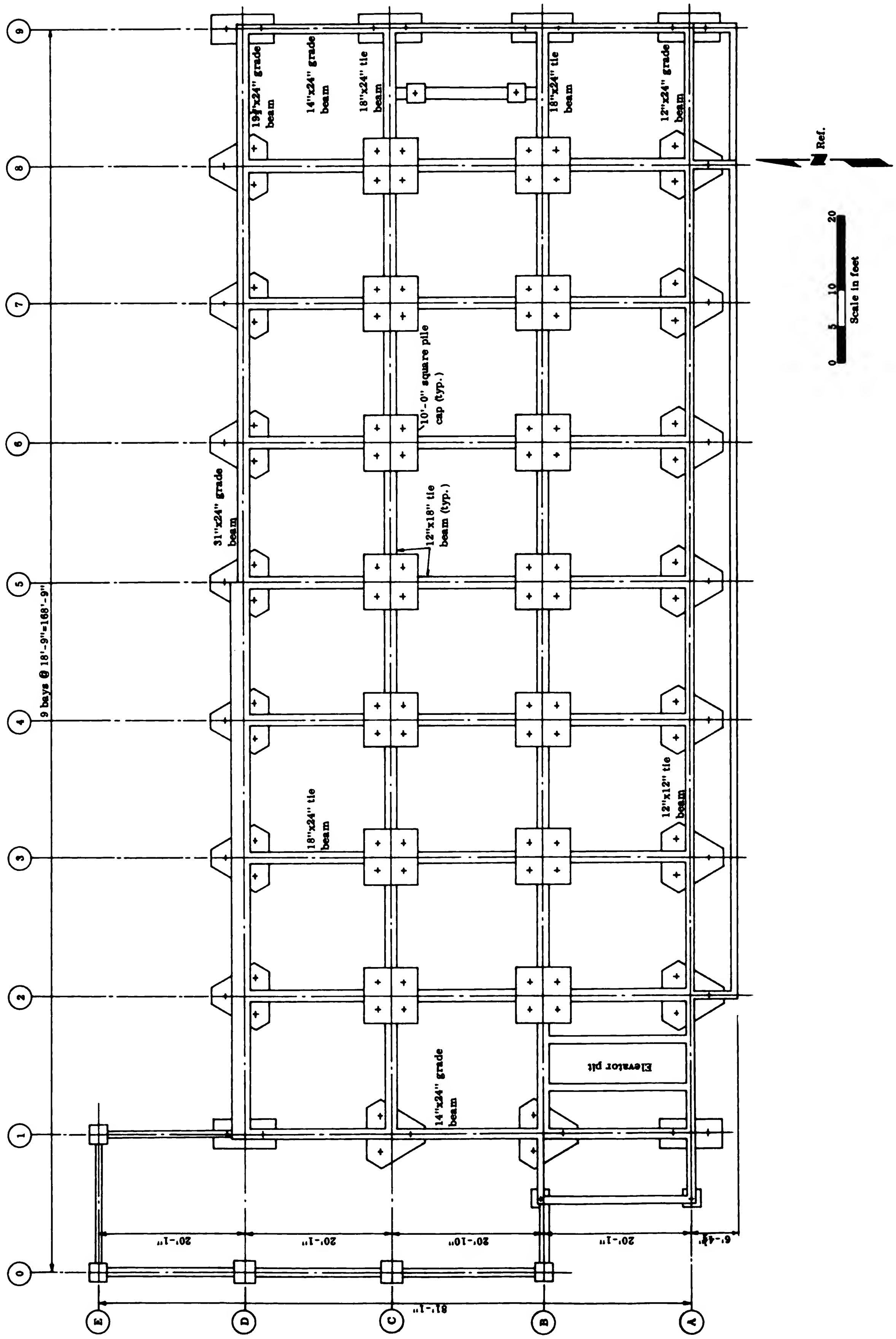


Figure 3.—Holiday Inn, Orion Avenue. Foundation plan.

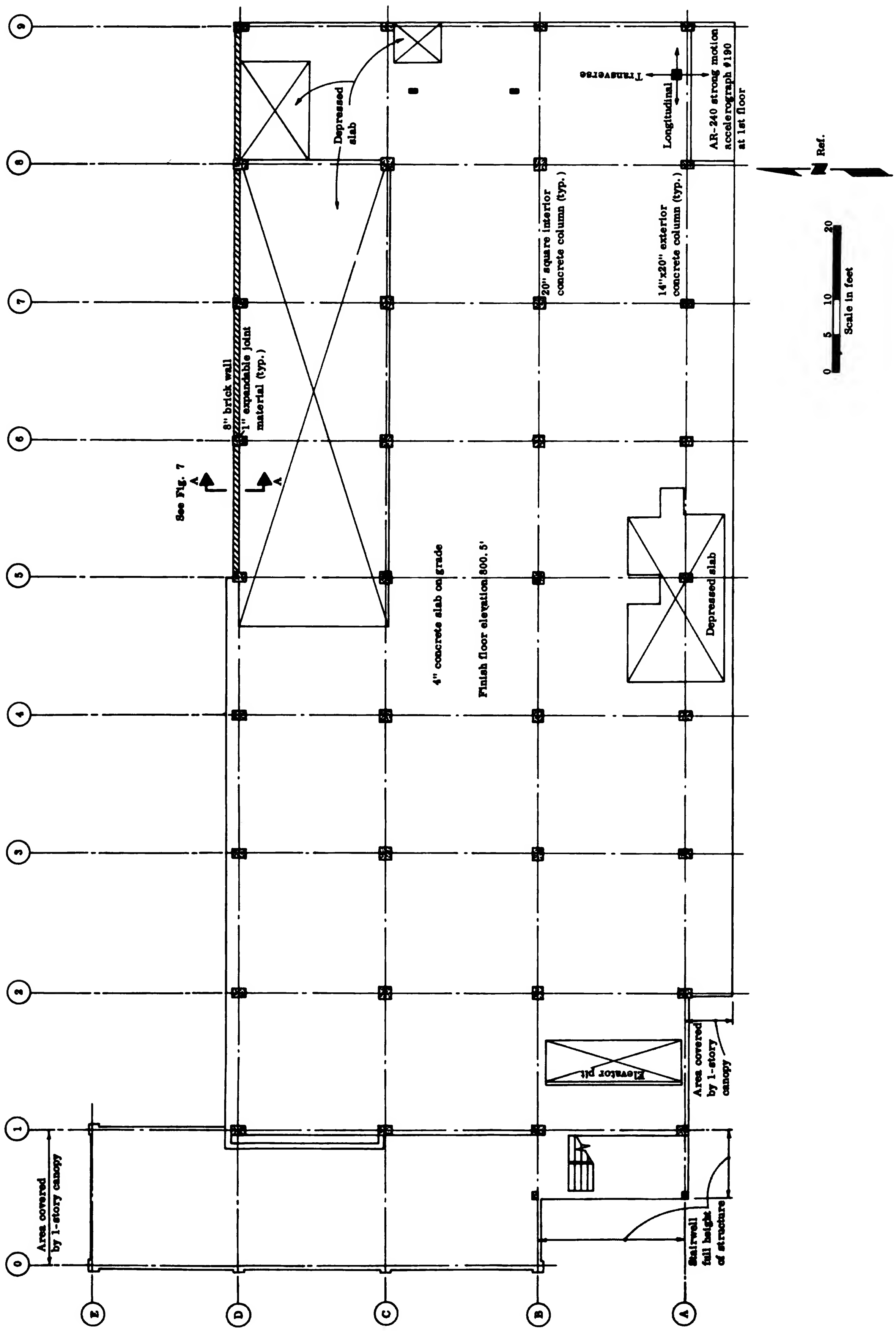


Figure 4.—Holiday Inn, Orion Avenue. First-floor plan.

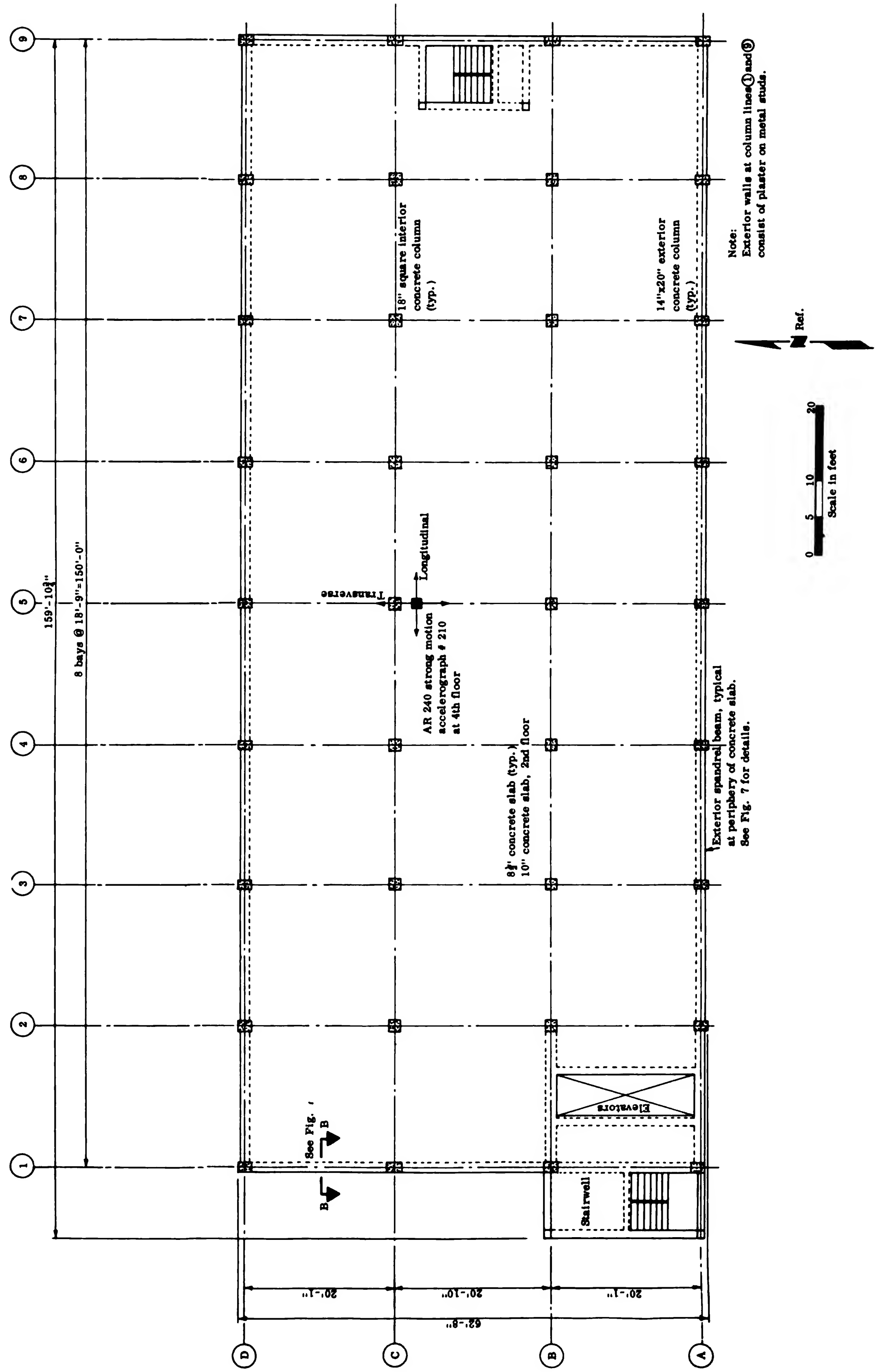


Figure 5.—Holiday Inn, Orion Avenue. Typical floor framing plan.

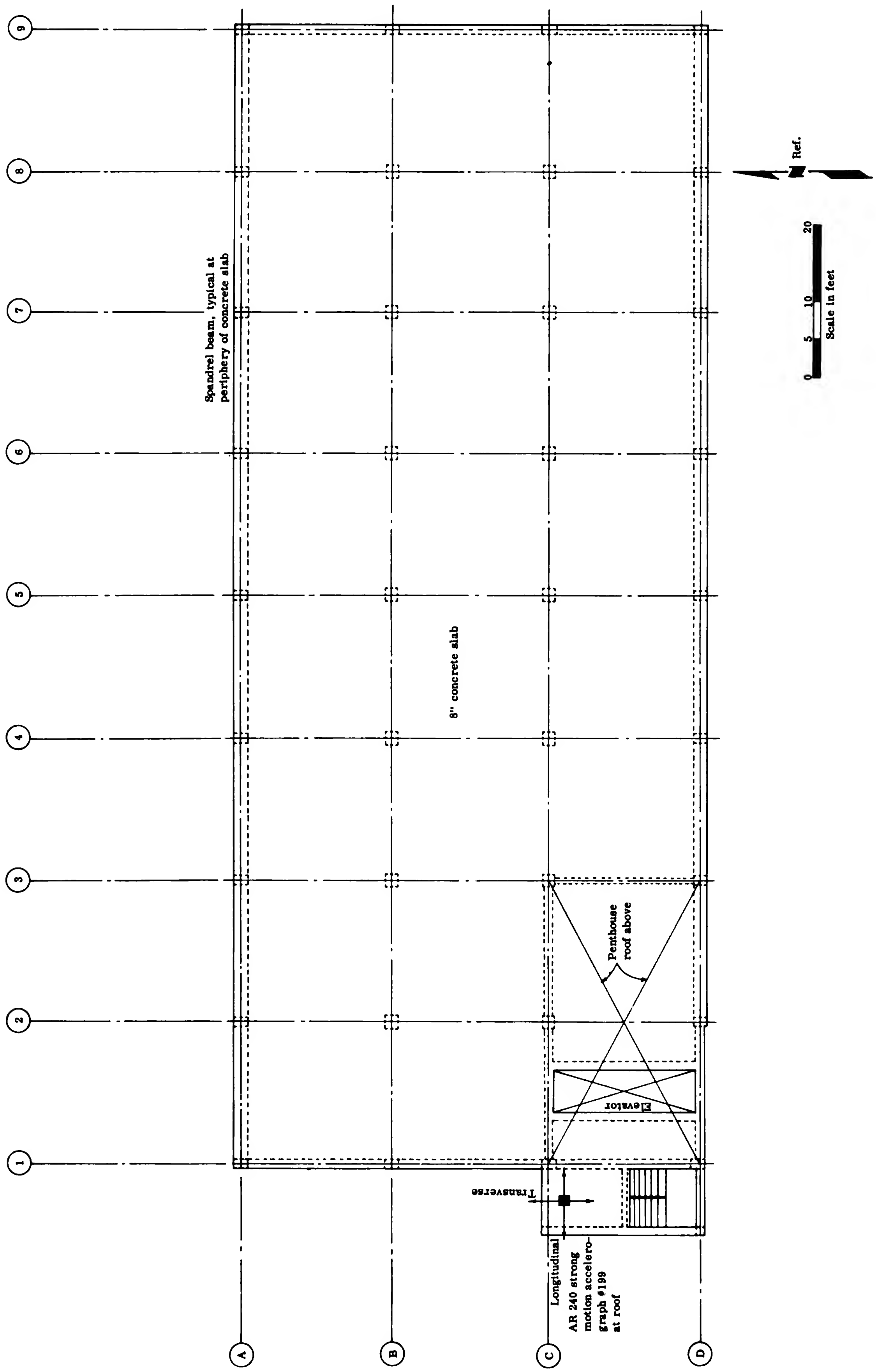


Figure 6.—Holiday Inn, Orion Avenue. Roof framing plan.

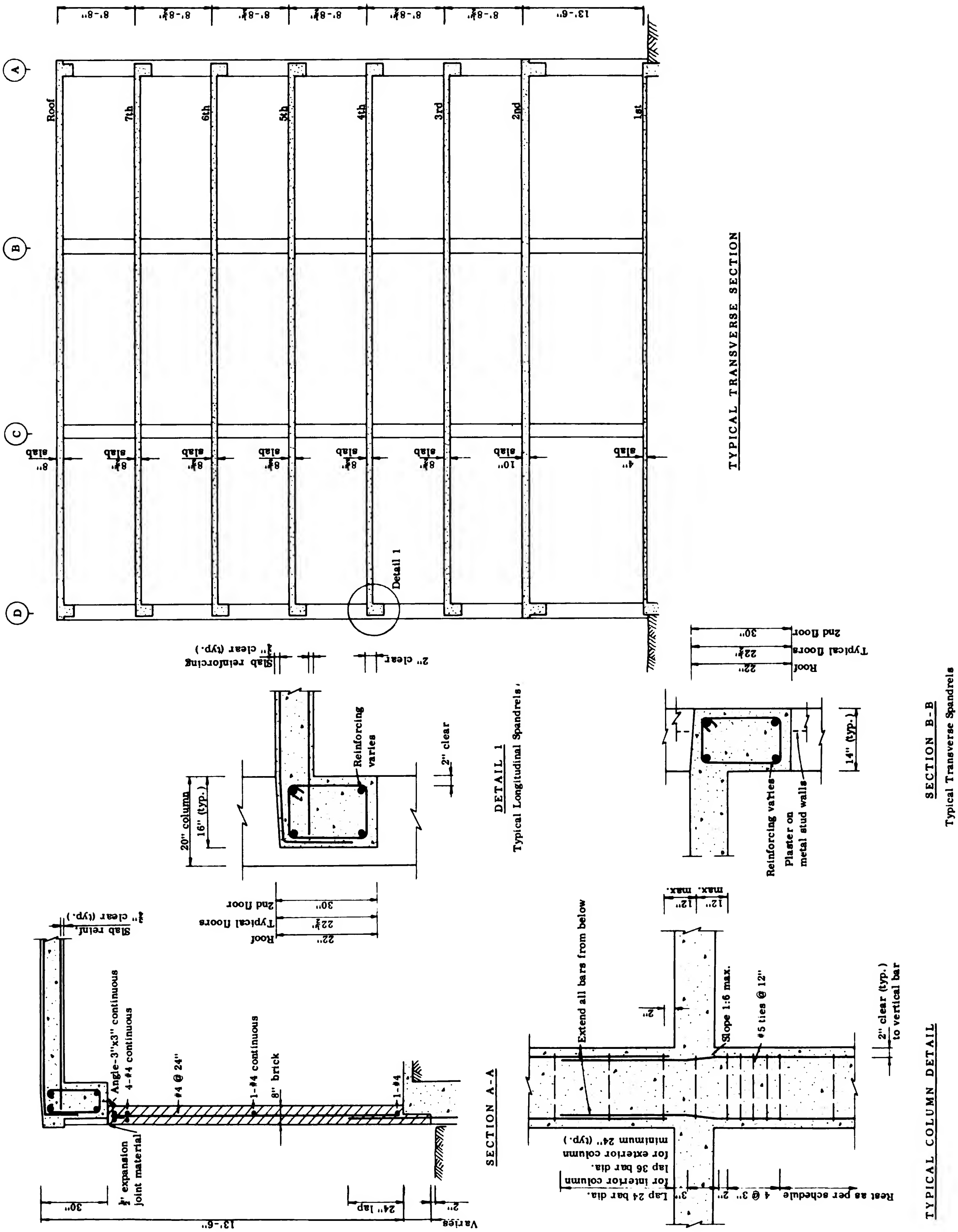


Figure 7.—Holiday Inn, Orion Avenue. Sections and details.

Lateral forces in each direction are resisted by the interior column-slab frames and by the exterior column-spandrel beam frames. The added stiffness afforded the exterior frames by the spandrel beams creates exterior frames that are roughly twice as stiff as interior frames.

With the exception of some light framing members supporting the stairway and elevator openings, the structure is essentially symmetrical. The participation of the nonstructural brick filler walls, and some exterior cement plaster, could cause some asymmetry for lateral motion in the longitudinal direction. However, it has been assumed that the effects of this would be minor.

Figure 8.—Holiday Inn, Orion Avenue. East end of north side of building. Crack at second-floor beam-column joint. John A. Blume & Associates photograph.

EARTHQUAKE DAMAGE

It cost approximately $145,000 to repair the damage caused by the San Fernando earthquake. This is roughly 11 percent of the initial construction cost of the building. Structural repair amounted to less than $2,000; the remainder was for nonstructural damage.

The structural repair consisted of patching the second-floor beam-column joint on the north side (east end) of the structure (figs. 8 and 9). Some structural distress appeared at some column pour joints located near the exterior beam soffits (fig. 10). Epoxy repaired the spalled concrete. Paint was applied to the areas where only flaking of paint occurred.

Nonstructural damage was extensive. Almost every guest room suffered some damage. About 80 percent of the repair costs was spent on drywall partitions, bathroom tile, and plumbing fixtures. The damage was most severe on the second and third floors and least severe at the sixth and seventh floors.

Some gypsum wallboard panels had to be replaced. Interior partitions required paint and new vinyl wall covering (fig. 11). Forty-five bathtubs (fig. 12) and 12 water closets had to be replaced. Bathroom tile had to be patched, grouted, or replaced in over half of the bathrooms (fig. 13). Spalling occurred at architectural concrete attached to structural concrete columns at the ground floor (fig. 14). Exterior cement plaster spalled and cracked. Windows in every room required some alignment and caulking, although none needed replacing. Doors needed adjustment.

The damage repair costs averaged approximately $2.30 per square foot of floor space.

Figure 9.—Holiday Inn, Orion Avenue. Closeup view of figure 8. John A. Blume & Associates photograph.

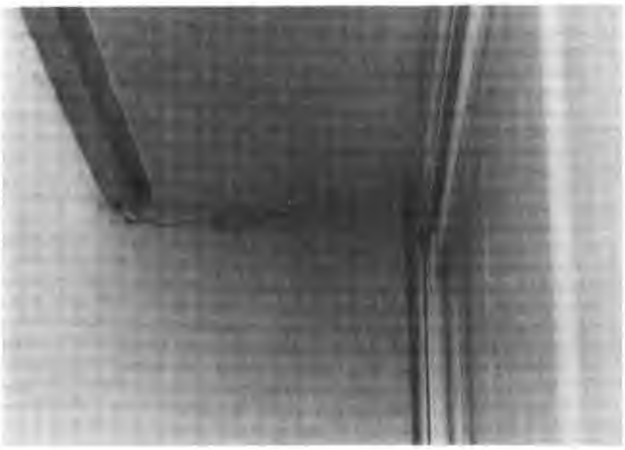


Figure 10.—Holiday Inn, Orion Avenue. Exterior spandrel beam-to-column connection. Shallow cracking at column pour joint. General Adjustment Bureau photograph.

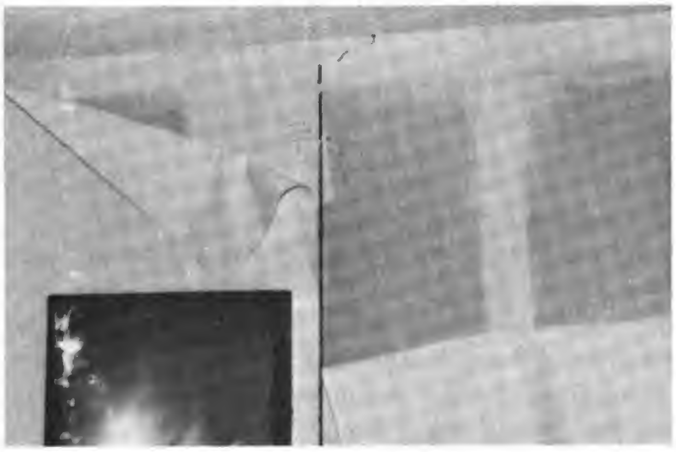


Figure 11.—Holiday Inn, Orion Avenue. Break in gypsum wallboard on second floor. John A. Blume & Associates photograph.

Figure 12.—Holiday Inn, Orion Avenue. Damage to bathtub at second floor. John A. Blume & Associates photograph.

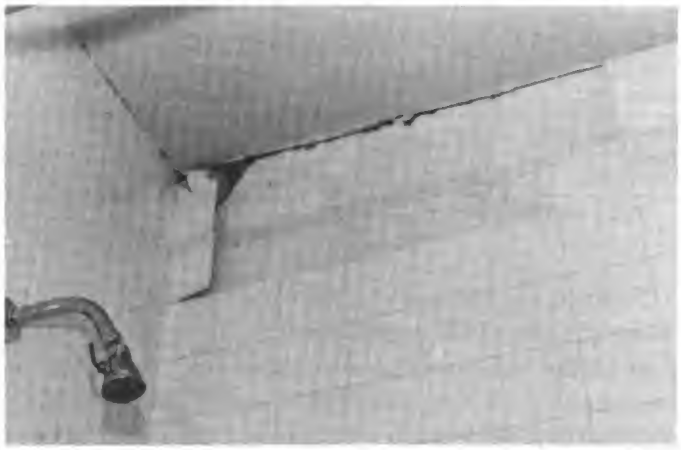


Figure 13.—Holiday Inn, Orion Avenue. Damage to tile above bathtub on second floor. John A. Blume & Associates photograph.

Figure 14.—Holiday Inn, Orion Avenue. Spalling of architectural concrete at first-floor column, north side. John A. Blume & Associates photograph.

RECORDED EARTHQUAKE RESPONSE

Motion caused by the San Fernando earthquake was recorded by Earth Sciences AR–240 strong-motion accelerographs located at the roof, fourth floor, and first floor (ground level). At each location (figs. 4, 5, and 6), motion was recorded along the three principal axes, parallel to the long direction of the building (longitudinal), parallel to the short side of the building (transverse), and vertically. Approximately 40 seconds of motion was recorded for each component of motion at each location. Table 2 gives the peak measured accelerations and their times of occurrence for each recording. These values were obtained from a digitized listing of the records.

Table 2.—Peak recorded accelerations [1]

Station	Transverse (north/south) component	Longitudinal (east/west) component	Vertical component
Roof (8th level)	0.406*g* at 9.9 sec	0.327*g* at 9.2 sec	0.24*g* at 3.60 sec.
4th floor	0.203*g* at 9.1 sec	0.253*g* at 7.9 sec	0.24*g* at 3.47 sec.
1st floor	0.251*g* at 12.5 sec	0.134*g* at 9.0 sec	0.18*g* at 3.62 sec.

[1] From digitized listing.

Figures 15, 16, and 17 were plotted by computer using the digitized records. From a visual examination of the longitudinal and transverse directions of the fourth-floor and roof-level records, the following observations were made: In the first 6 seconds of motion, the apparent fundamental period in each direction was roughly 0.7 second, but at about 9 seconds, the fundamental period appeared to be about 1.5 seconds. This leads to the hypothesis that the elastic limits of some elements in the structure were exceeded between 6 and 9 seconds after the start of motion; thereafter, the structure responded periodically in an inelastic manner. This hypothesis formed the basis for the analysis discussed later in this report.

From the digitized first-floor ground motion record, response spectra were determined for various percentages of critical damping. Figures 18 and 19 show the response spectra for the transverse and longitudinal directions at 2- and 10-percent critical damping. Response spectra for vertical motion (figs. 28, 29, and 30) are discussed in a later section. Response spectra have been plotted on four-way log paper to aid the reading of either pseudo-absolute acceleration, pseudo-relative velocity, or relative displacement values.

MATHEMATICAL MODELING

The description of analytical procedures outlined the general mathematical modeling procedure. For the Holiday Inn, gross concrete sectional properties (I_o and A_g) were calculated for columns, beams, and slabs. These values were then adjusted, as indicated in table 3, to compensate for the effects of composite beam and slab section, effective slab width, reinforcing steel, the changes of modulus of elasticity, and the differences in effective member lengths. The mathematical models (figs. 20 and 21) include column widths and clear spans of beams. In the vertical direction, the structure was dimensioned for the clear column lengths between slabs and for the slab thickness. Therefore, adjustments only had to

Table 3.—Member stiffness properties

Member	Moment of inertia	Shear area	Cross sectional area
Spandrel beams	$1.5\ C_eI_o$		
Slabs	$1.2\ C_eI_o$		
Interior columns	$C_sC_eI_o$	$5/6\ C_eA_g$	C_eA_g
Exterior columns	$C_sC_eC_l^3I_o$	$5/6\ C_eC_lA_g$	C_eA_g

$C_e = \sqrt{E/3.3 \times 10^6}$ (See table 1 for values of E.)
C_l = Clear-story height between slabs divided by clear-column length between beams. (A value of 1.17 was used for the typical exterior columns.)
C_s = Ratio of moment of inertia of column, including effects of reinforcing steel, to I_o. For models TS1 and LS1, $C_s = 1.0$ because the reinforcement was not considered. For models TS2 and LS2, $C_s = 1.4$, an average of the minimum and maximum calculated values of C_s.

be made for the columns at the exterior frames to compensate for the depth of spandrel beams.

Because the study was limited to planar analyses, each mathematical model was two-dimensional, and the building was assumed to be symmetrical with no eccentricity between the center of mass and the center of rigidity. This assumption is considered to be reasonable because, as discussed earlier, this building is essentially symmetrical with the center of mass essentially coincident with the center of rigidity. In addition, the more rigid frames were positioned at all the extremities of the structure, which reduced the effects of accidental torsion. The assumption that the concrete floor slab acts as a rigid horizontal diaphragm is also considered to be reasonable because of the absence of any significant openings, and because the length-to-width ratio is approximately 2.5.

Tables 4 and 5 give descriptions of the various models used in the transverse and longitudinal directions. Models TS1 and LS1 represent the first trial runs. Because the resulting periods appeared long, column reinforcing was added to create models TS2 and LS2. This is justified because the columns are in compression, and therefore the full transformed section can be considered to be effective. Reinforcing was not added to the beams and slabs because these elements were considered cracked sections, which

Table 4.—Transverse direction mathematical models used in the analysis

Model	Fundamental period	Lateral force-resisting system	Purpose	Earthquake time interval	Number of modes	Applied viscous damping
	Seconds			*Seconds*		*Percent*
TS1	0.93	Bare structural frame	Periods and mode shapes			
TS2	.88	Increase column moments of inertia to include reinforcing steel.	Periods, mode shapes, and member forces from modal displacements.			
TS2–P	.84	Add partitions and plaster as equivalent diagonal struts.	Study influence of partitions			
TS2–PW	.54	Model plaster at ends as shear walls.	Study influence of exterior plaster acting as a shear wall.			
TD1	.70	TS2 adjusted for period, E = 5.1 × 10⁶ psi.	Dynamic analysis, preyield	0 to 6	3	10
TD2	1.60	TS2 adjusted for period, E = 1.0 × 10⁶ psi.	Dynamic analysis, postyield	0 to 24	3	10

Table 5.—Longitudinal direction mathematical models used in the analysis

Model	Fundamental period	Lateral force-resisting system	Purpose	Earthquake time interval	Number of modes	Applied viscous damping
	Seconds			*Seconds*		*Percent*
LS1	0.86	Bare structural frame	Periods and mode shapes			
LS2	.79	Increase column moments of inertia to include reinforcing steel.	Periods, mode shapes, and member forces from modal displacements.			
LS2–PW	.68	Add partitions, plaster, and brick.	Study effects of nonstructural elements.			
LS2–PW2	.64	Double stiffness of brick and add adjacent plaster 1st-floor wall.	Study effects of stiffening 1st-floor nonstructural elements.			
LD1	.70	LS2 adjusted for period, E = 4.2 × 10⁶ psi.	Dynamic analysis, preyield	0 to 6	3	5
LD2	1.50	LS2 adjusted for period, E = 0.91 × 10⁶ psi.	Dynamic analysis, postyield	0 to 24	3	10

would reduce the effective I_o. It was assumed that the increase due to reinforcement balances the decrease due to the effects of cracked sections.

The calculated periods of TS2 and LS2 were still longer than the 0.70-second period measured during the early portion of the earthquake record. The effects of nonstructural elements were added to the mathematical model by simulating partition elements in the form of diagonal struts. Equivalent elastic stiffness characteristics were obtained from both published (references 11 and 12) and unpublished laboratory test results.

This report does not detail an analytical approach for including the effects of partitions. However, the results of the analysis are presented to give a general approach toward including partitions in the mathematical model. In the transverse direction, the inclusion of partitions in model TS2–P reduced the period slightly, but not enough to match the recorded period. In model TS2–PW, additional stiffness was added by treating the plaster wall at each end as a solid shear wall, ignoring some effects of construction and expansion joints. This significantly affected the reduction of the periods of the structure. Apparently, the response of the structure during the first 6 seconds of the earthquake can be represented by a model about half way between models TS2–P and TS2–PW.

In the longitudinal model, LS2–PW, the inclusion of interior partitions, exterior plaster at the stair and elevator bay, and the four bays of brick wall at the first floor significantly reduced the period to nearly match the early recorded earthquake motion. Model LS2–PW2 was additionally stiffened by doubling the stiffness characteristics of the brick and by adding the participation of exterior plaster walls at the south canopy area of the first floor. This model becomes more relevant in the discussion of the Marengo Street structure (Building Report 30).

For the dynamic time-history analysis, the bare structural frame representations of models TS2 and LS2 were used as the basic mathematical model.

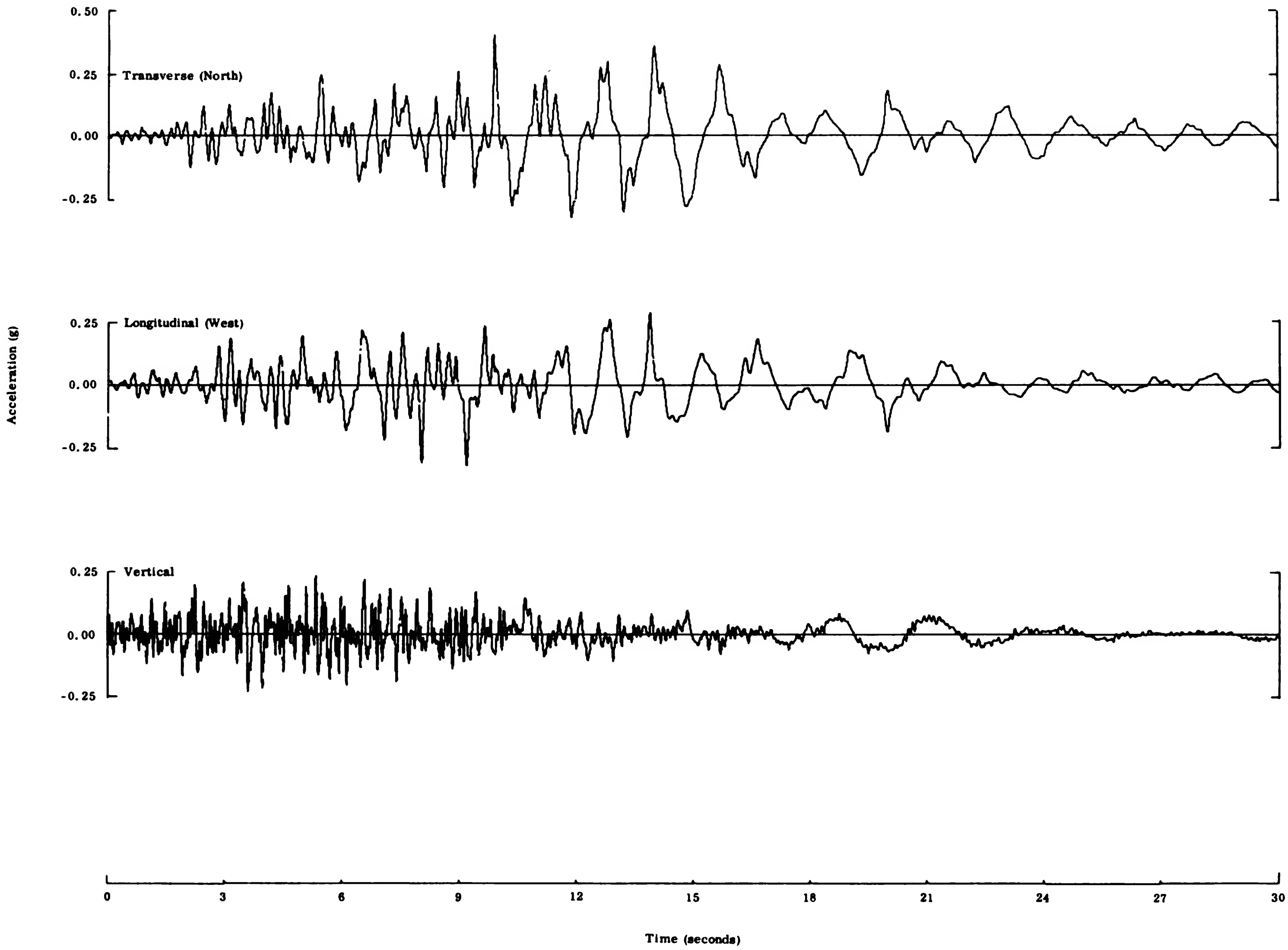


Figure 15.—Holiday Inn, Orion Avenue. Recorded acceleration at the roof level.

These models were adjusted to match the recorded fundamental periods by changing the value for the modulus of elasticity, E. In effect, this is the same as changing the values of I_o and A_g because the stiffness of the structure is dependent on the products of EI_o and A_gE. By using this method to adjust the natural periods of vibration, the characteristics of the calculated structural model are maintained. This assumes equally distributed softening or hardening effects between all structural elements. Although this actually may not be correct, it provides an economical alternative to a bilinear or inelastic procedure, which surpasses the scope of this report.

Models TD1 and LD1 represent the structure during the early portions of the earthquake. Models TD2 and LD2 represent the structure during the latter part of the record. The periods and damping values were obtained by analyses, correlating recorded and computed acceleration response.

RESULTS OF ANALYSIS

The previous section described the mathematical models developed for each principal direction. Each model was developed from a careful assessment of the anticipated actual stiffness characteristics and mass distribution of the structure. Review of the recorded motion records (figs. 15 and 16) indicated that the building, during approximately the first 6 seconds of the earthquake, responded at shorter periods than that indicated by the fundamental periods of the basic mathematical models, TS2 and LS2.

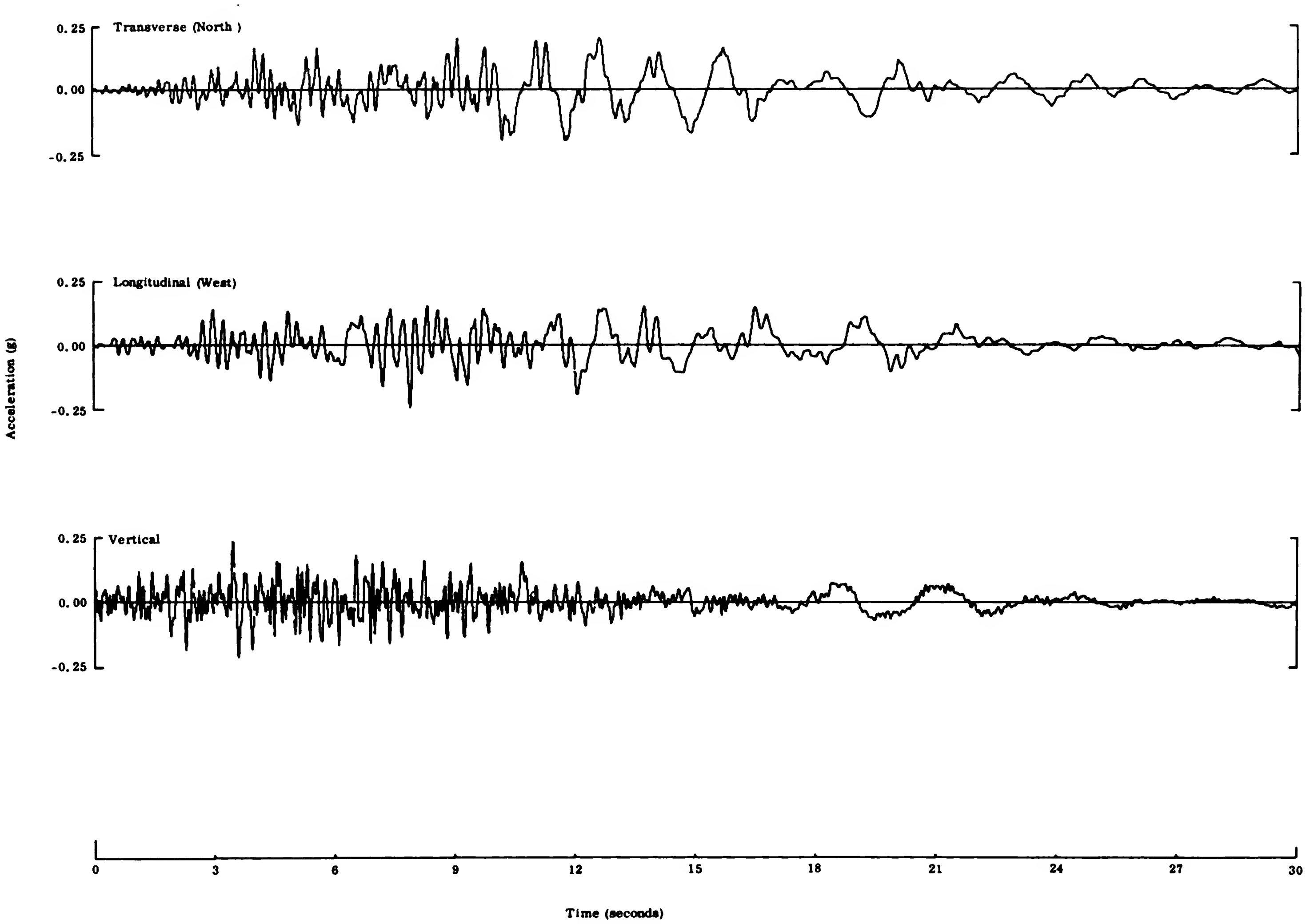


Figure 16.—Holiday Inn, Orion Avenue. Recorded acceleration at the fourth-floor level.

These differences appear to be due to the participation of nonstructural elements, such as drywall partitions, exterior plaster walls, and infill brick walls. For the latter part of the earthquake, the structure responded at longer periods than those indicated by the basic mathematical models. Apparently, these differences were due to a loss of stiffness caused by some structural members exceeding the elastic capacity.

Mode Shapes and Periods of Vibration

Mode shapes and periods of vibration were calculated for the first seven translational modes in both the transverse and longitudinal directions (tables 6 and 7). A review of these modes, in conjunction with the response spectra, shows that the higher modes are progressively less responsive to the earthquake. Therefore, the analysis only considered the first three translational modes in each direction. Figure 22 shows the masses, periods, and mode shapes for the three modes of models TS2 and LS2. The mode shapes are normalized at the roof to give the relative shapes between modes. Tables 6 and 7 show numerical values for the same models. These mode shapes and masses also apply to the transverse models, TD1 and TD2, and the longitudinal models, LD1 and LD2. Therefore, the periods for the higher modes of these models can be obtained by proportioning the higher mode periods to the fundamental periods in the same ratio as the TS2 and LS2 models.

Table 6.—Mode shapes and periods, transverse direction, model TS2

Mode number		1	2	3	4	5	6	7
Period of vibration (seconds)		0.880	0.288	0.164	0.106	0.073	0.055	0.046
Floor level	Mass	Mode shapes						
	kips-sec²/ft							
Roof	43.7800	0.0794	0.0747	0.0684	−0.0588	−0.0439	−0.0273	−0.0123
7th	45.3400	.0745	.0411	−.0040	.0501	.0768	.0703	.0382
6th	45.3400	.0666	−.0042	−.0644	.0635	−.0085	−.0740	−.0623
5th	45.3400	.0558	−.0471	−0.630	−.0309	−.0735	.0260	.0757
4th	45.3400	.0425	−.0718	−.0023	−.0740	.0469	.0399	−.0763
3d	45.3400	.0279	−.0697	.0604	.0052	.0500	−.0785	.0639
2d	56.8300	.0149	−.0467	.0677	.0676	−.0575	.0447	.0272
	Σ = 327.3100							
Effective V/W ÷ spectral acceleration		.83	.12	.04	.01	.00	.00	.00
Modal roof acceleration ÷ spectral acceleration		1.31	−.47	.24	−.11	.05	−.02	.00

Table 7.—Mode shapes and periods, longitudinal direction, model LS2

Mode number		1	2	3	4	5	6	7
Period of vibration (seconds)		0.791	0.266	0.156	0.104	0.076	0.060	0.052
Floor level	Mass	Mode shapes						
	kips-sec²/ft							
Roof	43.7800	0.0765	0.0719	0.0684	−0.0614	−0.0476	0.0305	0.0139
7th	45.3400	.0728	.0450	.0045	.0439	.0762	−.0731	−.0403
6th	45.3400	.0663	.0037	−.0599	.0702	−.0002	.0718	.0630
5th	45.3400	.0568	−.0388	−.0698	−.0211	−.0764	−.0222	−.0754
4th	45.3400	.0450	−.0673	−.0165	−.0774	.0417	−.0423	.0756
3d	45.3400	.0315	−.0722	.0515	−.0054	.0544	.0786	−.0634
2d	56.8300	.0191	−.0549	.0700	.0638	−.0530	−.0419	.0261
	Σ = 327.3100							
Effective V/W ÷ spectral acceleration		.86	.10	.03	.01	.00	.00	.00
Modal roof acceleration ÷ spectral acceleration		1.28	−.42	.20	−.09	.04	−.01	.00

The values given in tables 6 and 7 for the ratios of effective V/W to spectral acceleration are useful for spectral analysis. When multiplied by the appropriate spectral acceleration, these ratios (effective modal loads) will give the equivalent base shear coefficient. In the case of the first mode, this value can be compared with the UBC design coefficient, ZKC.

For example, take the value 0.86 from table 7. Multiply it by the spectral acceleration of 0.19*g* for a period of 0.79 second and a damping ratio of 10 percent (fig. 19). This gives a value of 0.16 for ZKC, which can be compared to the design code requirements of 0.04. This is discussed later (see also table 10).

Naturally, using different fundamental periods will yield different values. In a manner similar to that just described, spectral analysis can be aided by using the ratios for modal roof acceleration to spectral acceleration (tables 6 and 7). In these situations, one can calculate the peak roof acceleration for any mode.

Computed Floor Accelerations

The dynamic analyses procedure included calculation and plotting of accelerations for the roof and fourth-floor levels of the structure. These calculations were made by using the recorded first-floor motion, the SMIS computer program, the measured fundamental periods, and the calculated mode shapes and modal period relationships from models TS2 and LS2. The paper on analytical procedures in this volume describes these procedures.

Figures 23 and 24 show both calculated and recorded acceleration time histories. In each group of

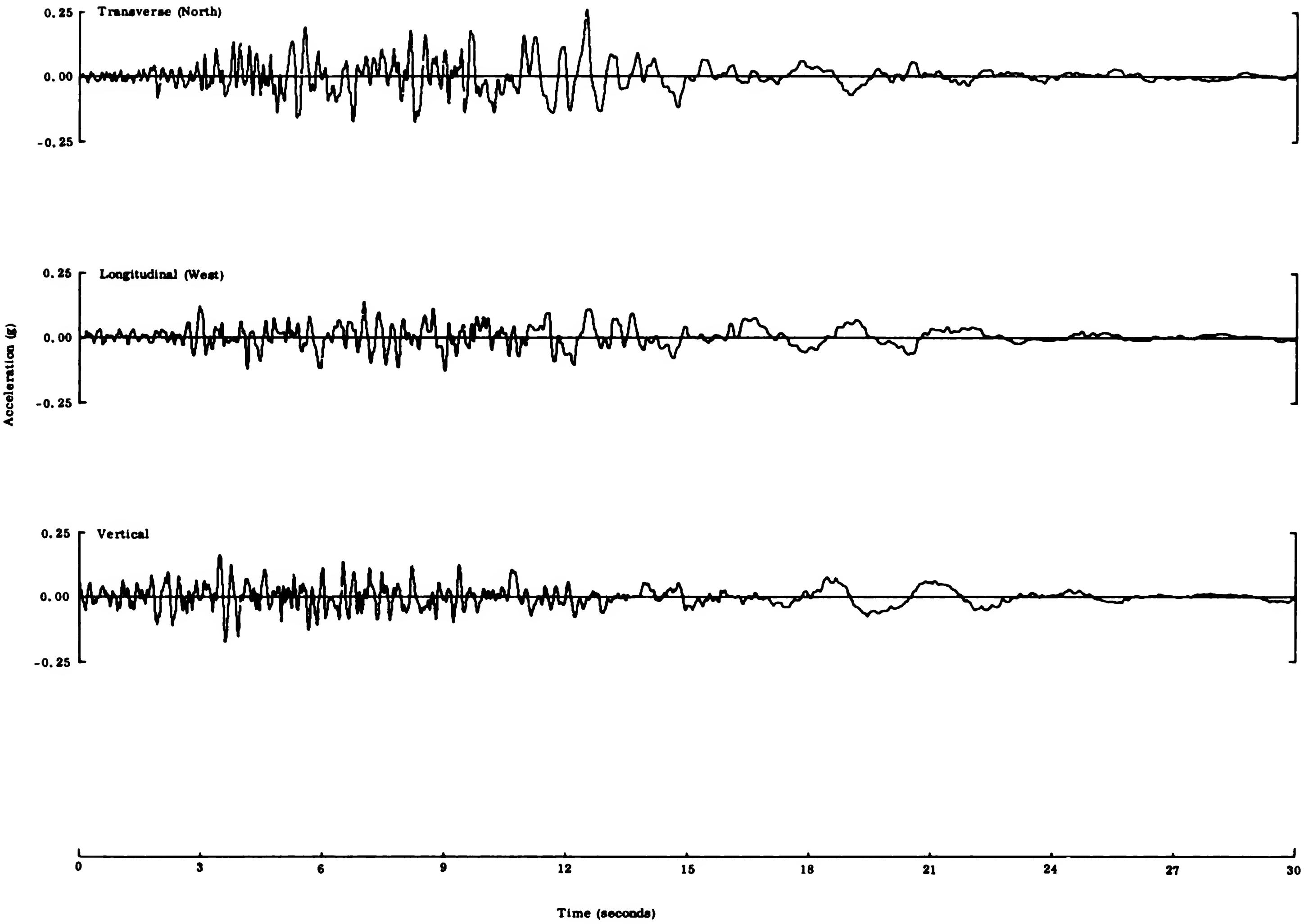


Figure 17.—Holiday Inn, Orion Avenue. Recorded acceleration at the ground level.

three plots, the center plot is the recorded acceleration, the top plot is the calculated acceleration for comparison with the early portion of the record (first 6 seconds), and the bottom plot is the calculated acceleration for comparison with the latter part of the earthquake record (after 9 seconds).

Tables 8 and 9 compare recorded and computed peak roof and fourth-floor accelerations. The FRMDYN model data show the peak maximum values for the combined first three modes, the time of occurrence, and the contribution of each mode. The response spectra analysis shows the peak value for each mode of the structure, assuming it had remained elastic during 40 seconds of motion. These peaks do not necessarily occur simultaneously. The sum of the three modes indicates what the maximum value could theoretically be if all three modes peaked at the same time. The root-mean-square (RMS) indicates a statistical mean value for the combination of the three modes. The peak measurement values and their times of occurrence were obtained from the digitized listing.

For the shorter period models, the response spectra analysis is invalid for comparison with the FRMDYN data because the peak responses of the response spectra occur after the 6-second duration of the FRMDYN model had ended. However, the response spectra summations and root-mean-square accelerations can be compared with the peak accelerations of the calculated time-history roof and fourth-floor plots shown in figures 23 and 24.

In tables 8 and 9, the FRMDYN model results reasonably agree with the measured results. Differences can be attributed to the phase relationship between

Table 8.—Maximum transverse accelerations and displacements at the roof and the fourth floor

	Mode(s)	Period	Damping	Accelerations		Displacements	
				Roof	4th	Roof	4th
		Seconds	*Percent critical*	*g*	*g*	*Feet*	*Feet*
FRMDYN model TD1 calculated for first 6 seconds of motion.	1st	0.70	10	0.24	0.066	0.093	0.050
	2d	.23	10	.12	.091	.005	−.005
	3d	.13	10	−.06	.001	−.001	.000
	3 modes			.315	.160	.096	.045
	[time] [1]			[5.40]	[4.05]	[5.40]	[5.40]
Response spectrum analysis, FRMSTC model TS2 with TD1 periods, first 40 seconds of motion.	1st	.70	10	.65	.35	.26	.14
	2d	.23	10	.21	.20	.01	.01
	3d	.13	10	.08	.00	.00	.00
	SUM			.94	.55	.27	.15
	RMS [2]			.69	.40	.26	.14
Peak measurement during first 6 seconds	All			.245	.167		
	[time] [1]			[5.45]	[4.05]		
FRMDYN model TD2 calculated for first 24 seconds of motion.	1st	1.6	10	.32	.10	.66	.312
	2d	.52	10	.24	.19	.05	.001
	3d	.30	10	.00	.00	.00	.000
	3 modes			.561	.283	.712	.313
	[time] [1]			[12.88]	[12.63]	[12.88]	[15.13]
Response spectrum analysis, FRMSTC model TS2 with TD2 periods, first 40 seconds of motion.	1st	1.6	10	.33	.18	.64	.35
	2d	.52	10	.23	.22	.05	.05
	3d	.30	10	.13	.00	.01	.00
	SUM			.69	.40	.70	.40
	RMS [2]			.42	.28	.64	.35
Peak measurement in 40 seconds	All			.406	.203		
	[time] [1]			[9.9]	[9.1]		

[1] Brackets indicate time from start of motion (expressed in seconds). [2] Root-mean-square.

Table 9.—Maximum longitudinal accelerations and displacements at the roof and the fourth floor

	Mode(s)	Period	Damping	Accelerations		Displacements	
				Roof	4th	Roof	4th
		Seconds	*Percent critical*	*g*	*g*	*Feet*	*Feet*
FRMDYN model LD1 calculated for first 6 seconds of motion.	1st	0.70	5	0.14	0.08	0.055	0.03
	2d	.24	5	.06	.03	.00±	.00±
	3d	.14	5			.00±	.00±
	3 modes			.201	.118	.055	.030
	[time] [1]			[5.00]	[4.90]	[5.00]	[4.95]
Response spectrum analysis, FRMSTC model LS2 with LD1 periods, first 40 seconds of motion.	1st	.70	5	.33	.19	.13	.08
	2d	.24	5	.15	.14	.01	.01
	3d	.14	5	.06	.01	.00	.00
	SUM			.54	.34	.14	.09
	RMS [2]			.37	.24	.13	.08
Peak measurement during first 6 seconds	All			.199	.146		
	[time] [1]			[5.00]	[4.42]		
FRMDYN model LD2 calculated for first 24 seconds of motion.	1st	1.5	10	.24	.151	.46	.254
	2d	.50	10	.06	.028	−.01	.007
	3d	.29	10	−.02	.000	.00	.000
	3 modes			.278	.181	.463	.262
	[time] [1]			[12.50]	[12.38]	[12.50]	[12.38]
Response spectrum analysis, FRMSTC model LS2 with LD2 periods, first 40 seconds of motion.	1st	1.5	10	.26	.15	.46	.27
	2d	.50	10	.11	.10	.02	.02
	3d	.29	10	.06	.01	.00	.00
	SUM			.43	.26	.48	.29
	RMS [2]			.29	.18	.46	.27
Peak measurement in 40 seconds	All			.327	.253		
	[time] [1]			[9.2]	[7.9]		

[1] Brackets indicate time from start of motion (expressed in seconds). [2] Root-mean-square.

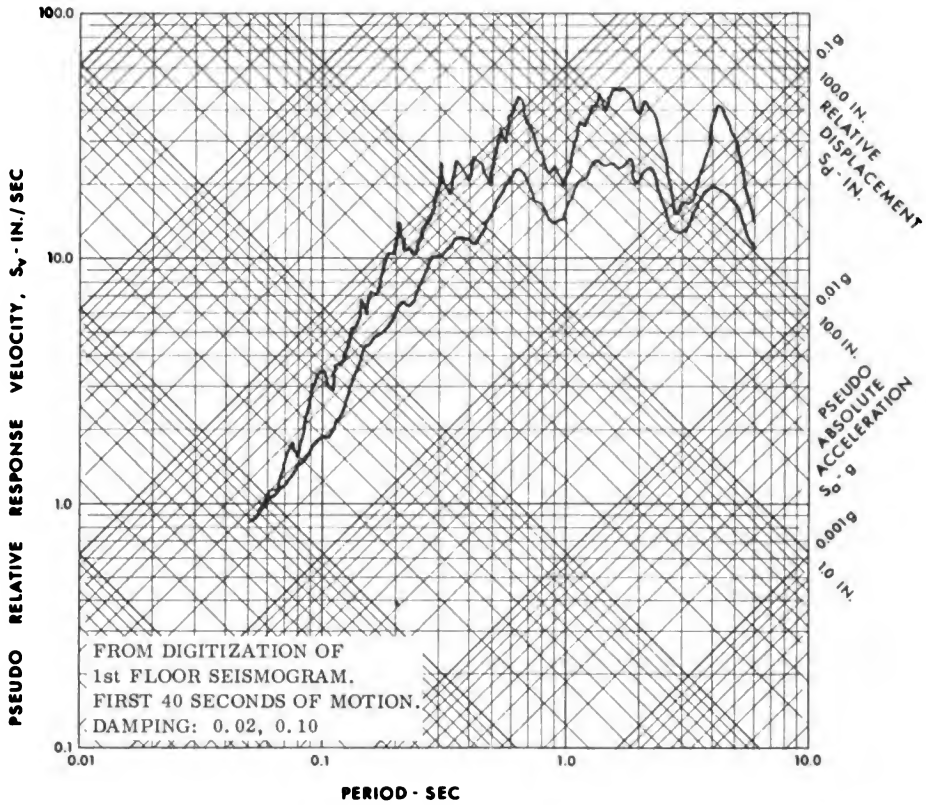


Figure 18.—Holiday Inn, Orion Avenue. Transverse response spectra.

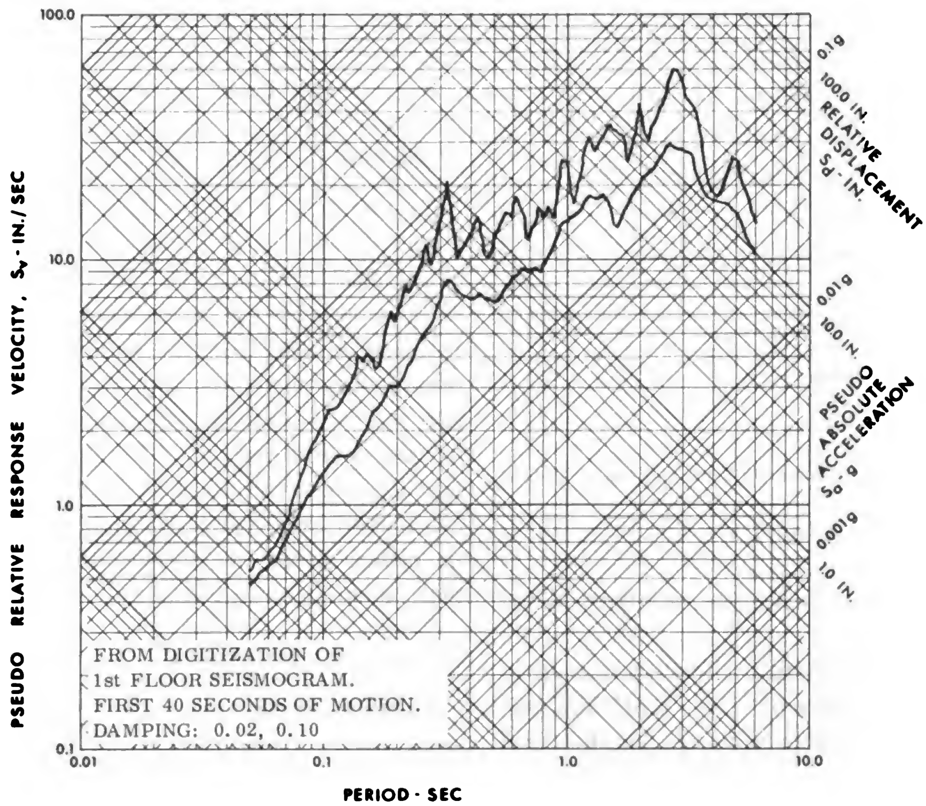


Figure 19.—Holiday Inn, Orion Avenue. Longitudinal response spectra.

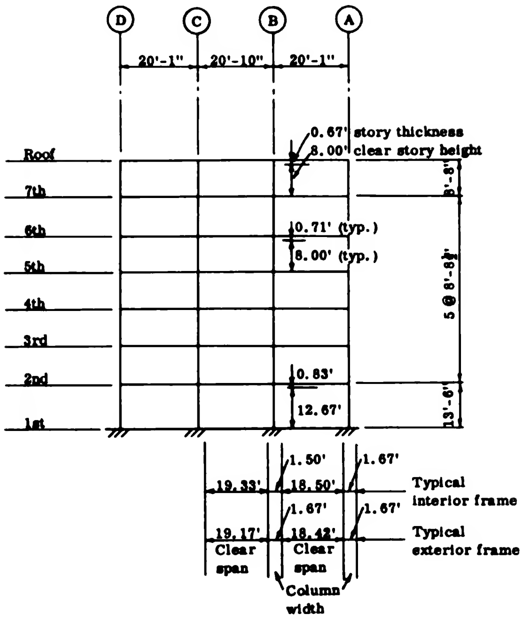


Figure 20.—Holiday Inn, Orion Avenue. Typical transverse frame.

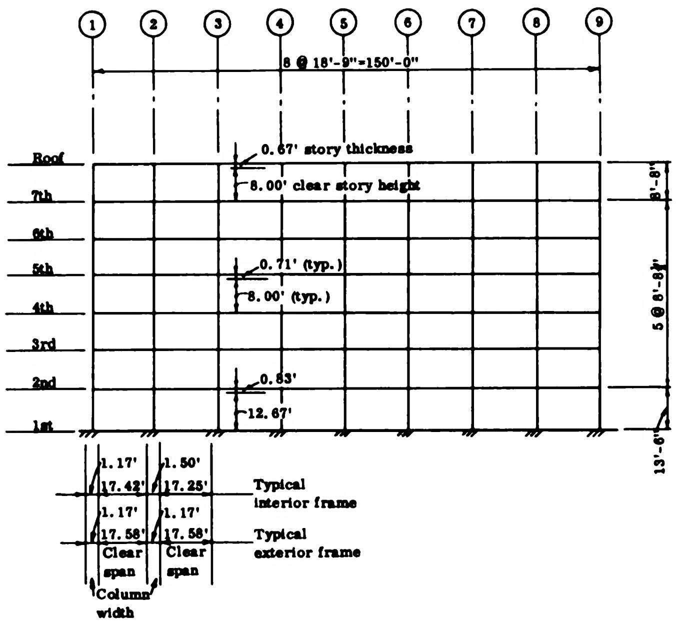


Figure 21.—Holiday Inn, Orion Avenue. Typical longitudinal frame.

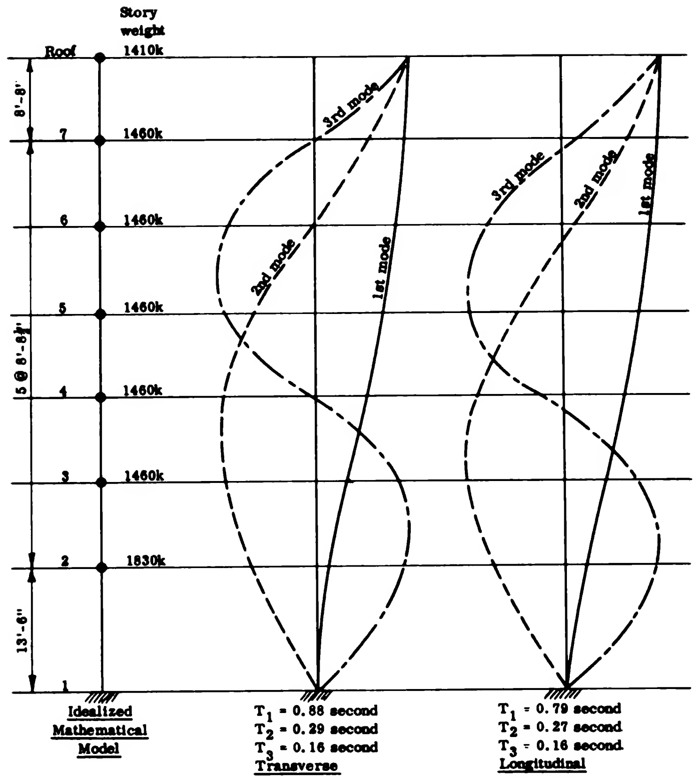


Figure 22.—Holiday Inn, Orion Avenue. Calculated periods and mode shapes.

different modes. This affects the summation of modal contributions.

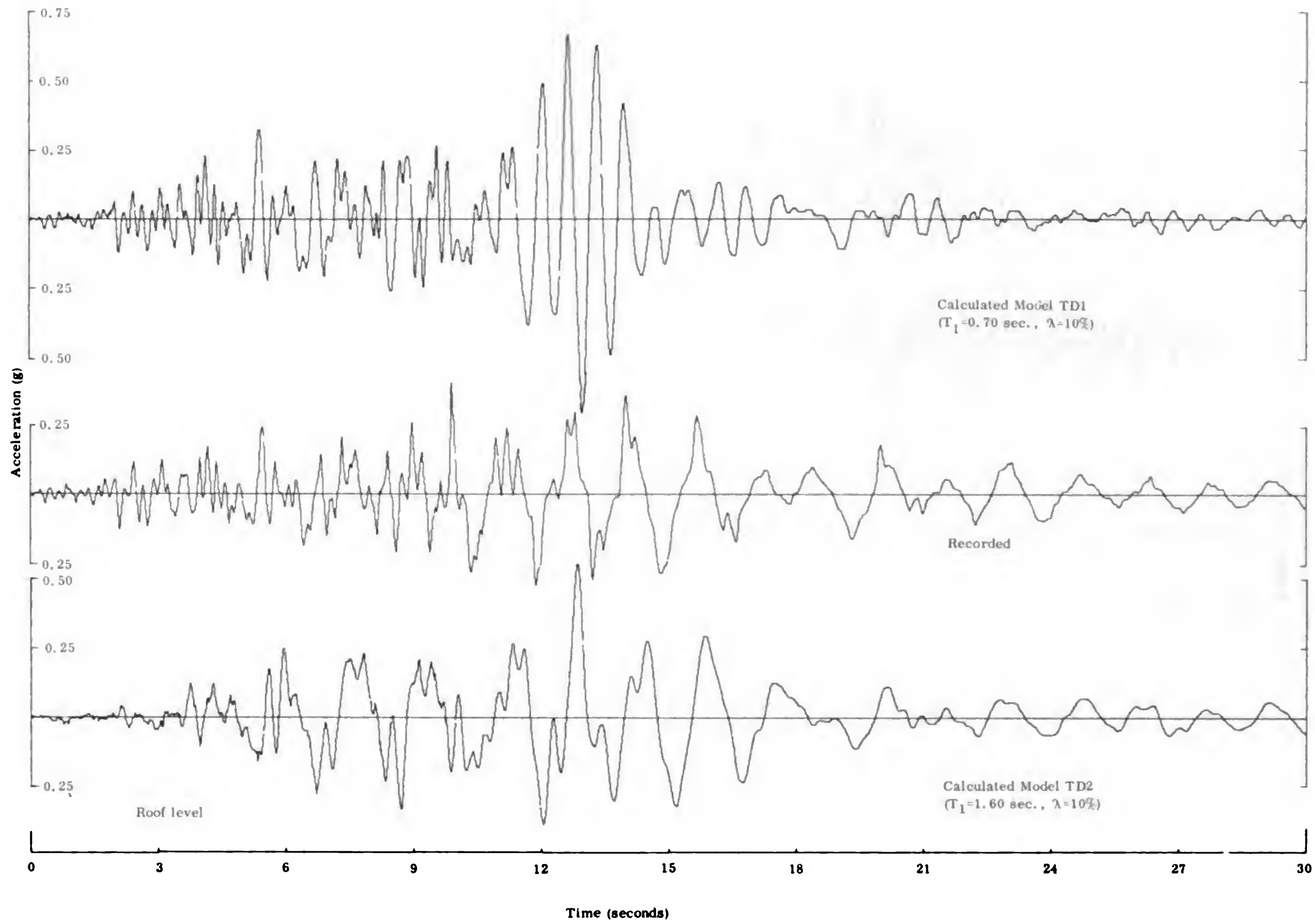
For the longer period models, the response spectra results can be compared with the FRMDYN and measured results. The FRMDYN results for the combined three modes and the peak measured results will be equal to or greater than the first-mode response spectra results, less than the summation of the response spectra results, and greater than or less than the root-mean-square response spectra results.

Maximum Building Displacements

Figures 25a and 26a show envelopes of maximum total displacement. These represent the peak displacement of each story with respect to the first floor. Not all of the maximums occurred simultaneously. For the shorter period models (TD1 and LD1), maximums occurred approximately 5 seconds after the start of motion. Maximums occurred for the longer period models (TD2 and LD2) beyond 12 seconds after the start of motion. Times for maximum roof and fourth-floor displacements are shown in tables 8 and 9. Figures 25a and 26a also show maximum interstory drifts.

Maximum Story Forces

Figures 25b and 26b plot the maximum horizontal story forces from the dynamic analysis and corresponding code values. For each level, story forces were determined as the product of the story mass times the maximum absolute story acceleration.



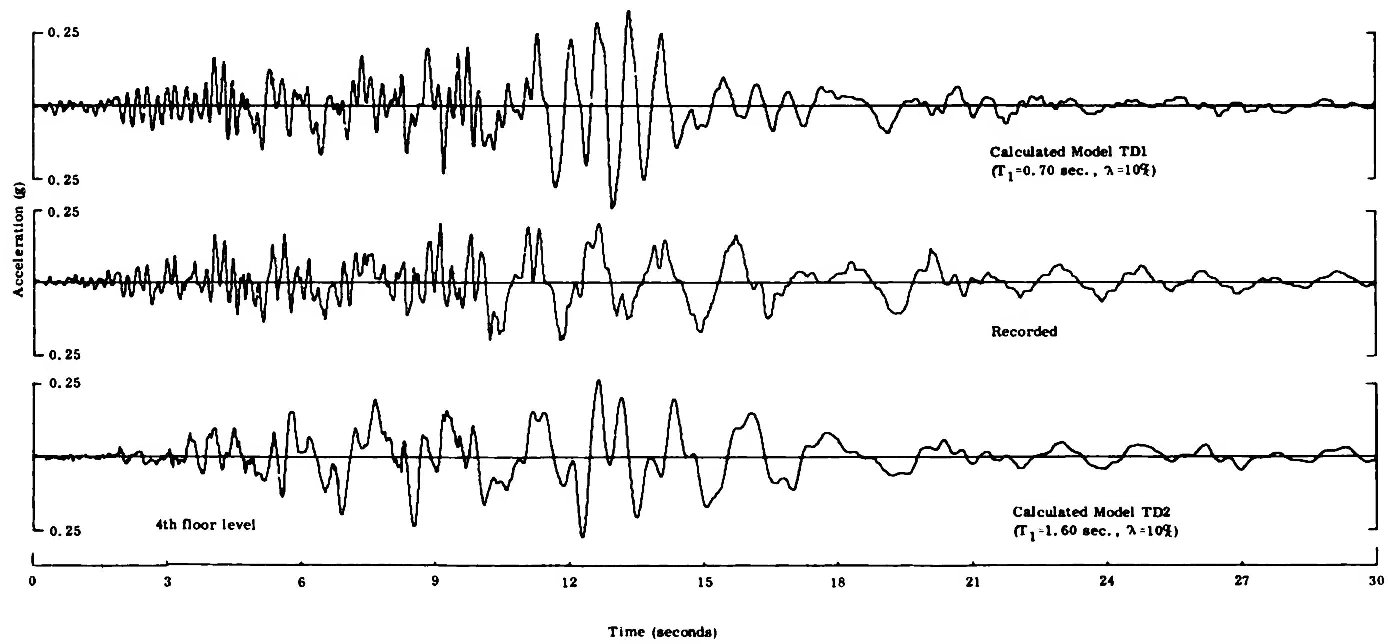


Figure 23.—Holiday Inn, Orion Avenue. Transverse calculated and recorded accelerations.

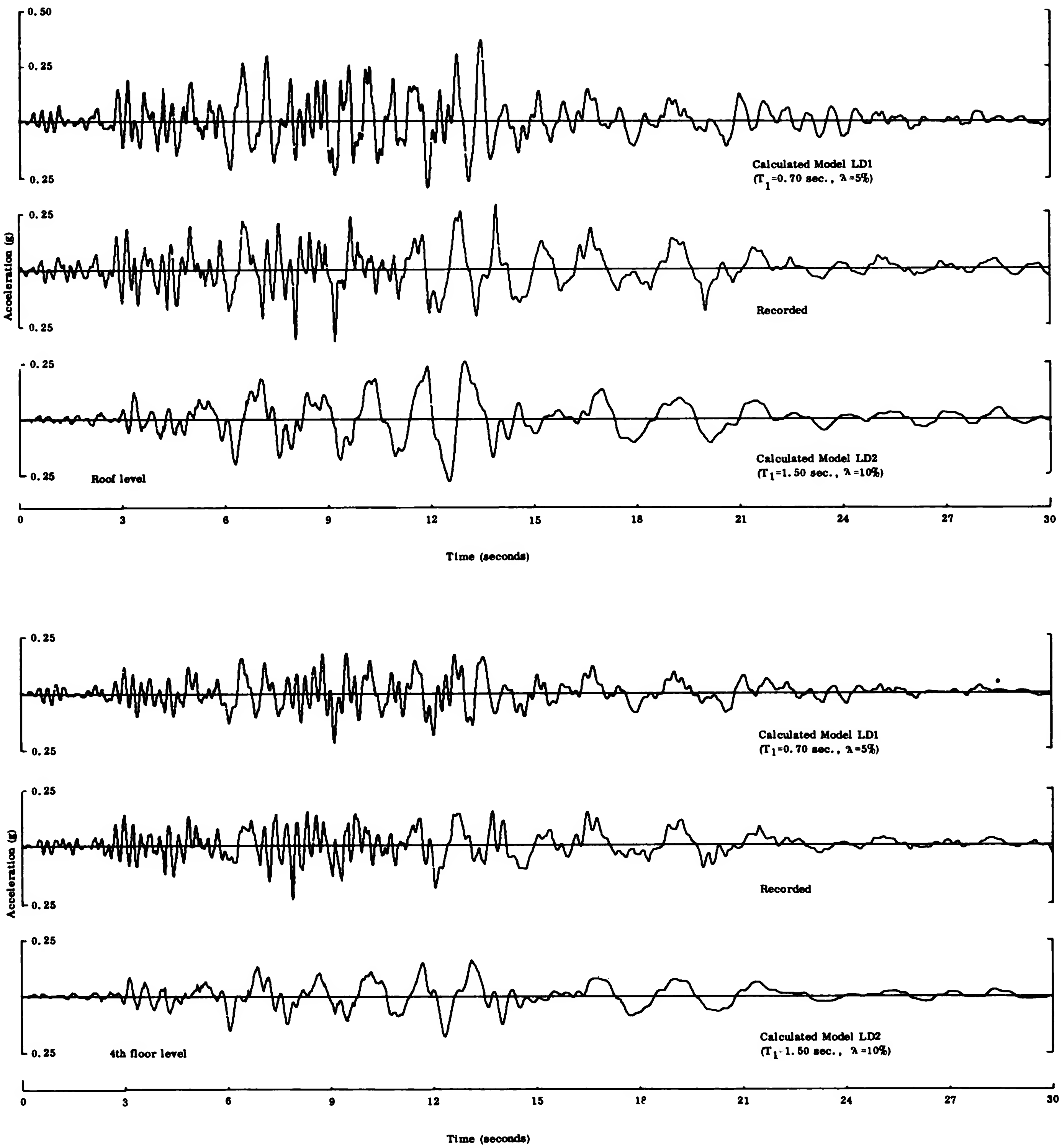


Figure 24.—Holiday Inn, Orion Avenue. Longitudinal calculated and recorded accelerations.

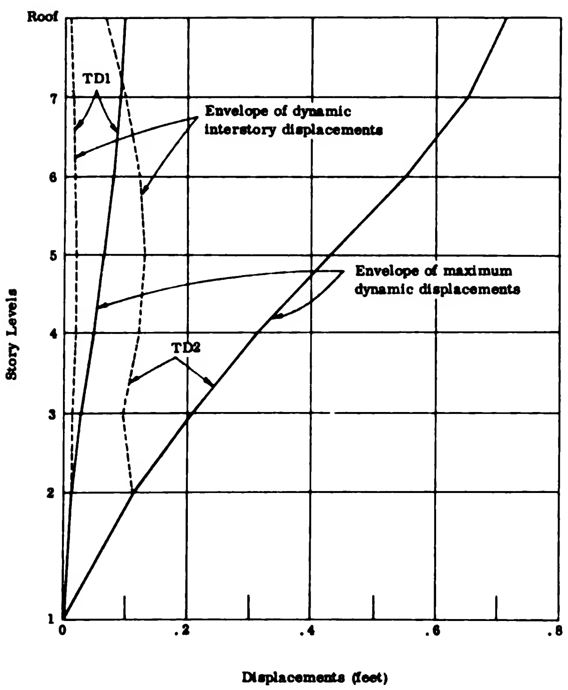


Figure 25a. TOTAL BUILDING DISPLACEMENTS AND INTERSTORY DISPLACEMENTS

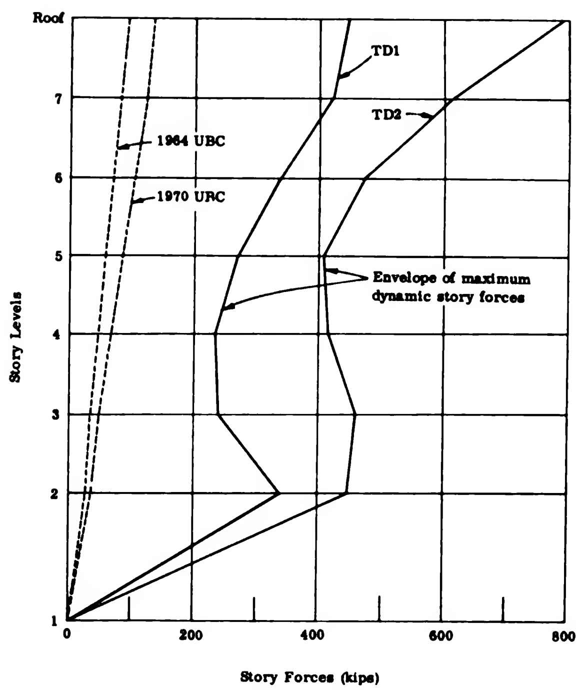


Figure 25b. MAXIMUM STORY FORCES

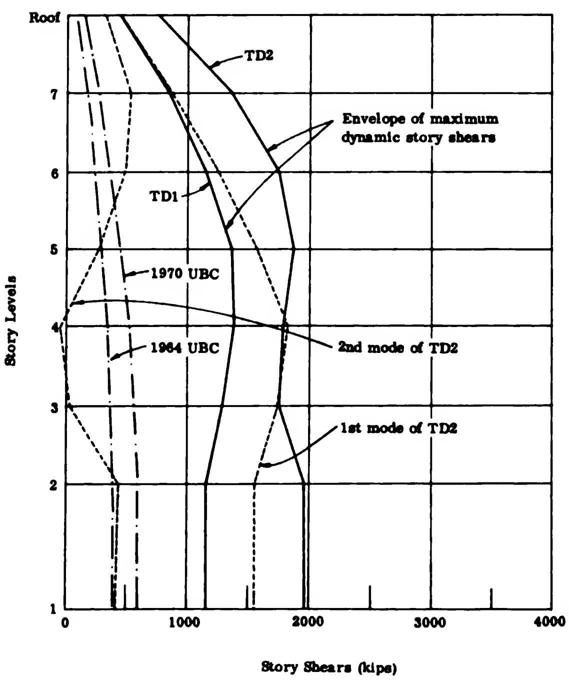


Figure 25c. MAXIMUM STORY SHEARS

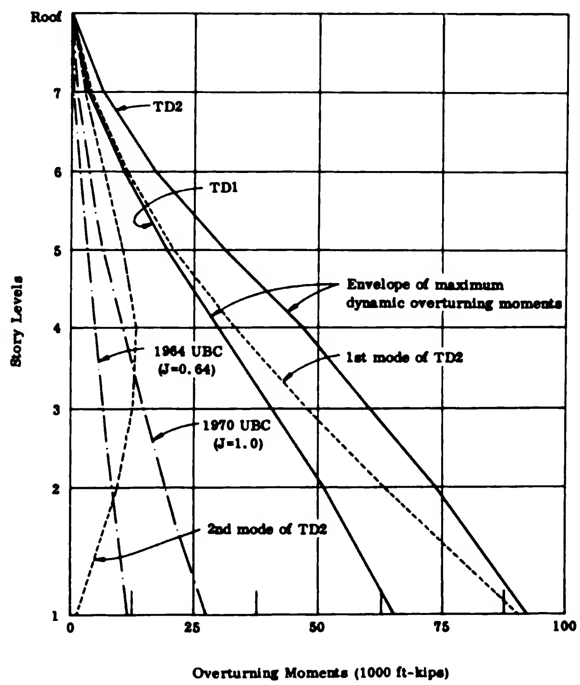


Figure 25d. MAXIMUM OVERTURNING MOMENTS

Figure 25.—Holiday Inn, Orion Avenue. Dynamic response and design code values for transverse (north-south) direction.

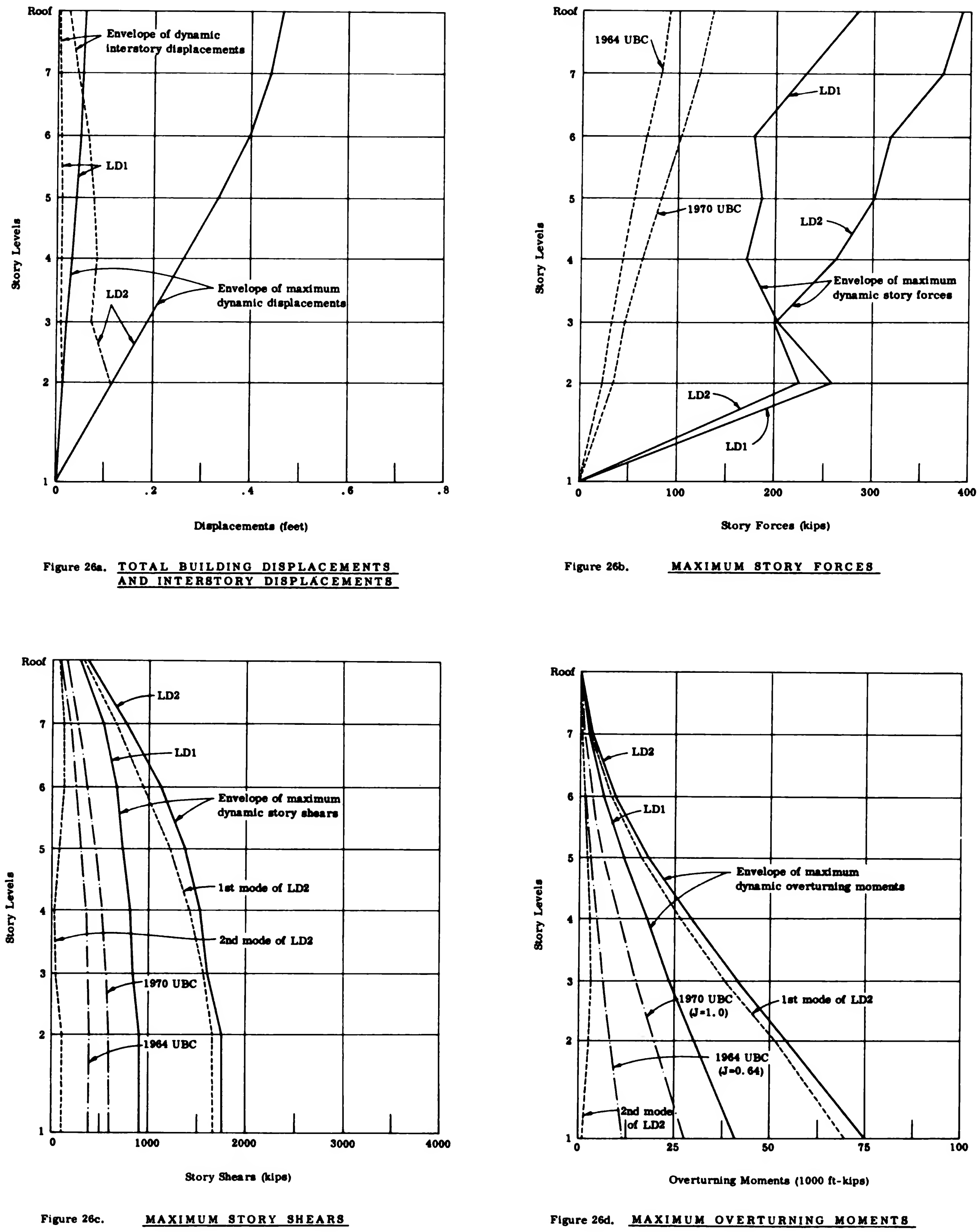


Figure 26.—Holiday Inn, Orion Avenue. Dynamic response and design code values for longitudinal (east-west) direction.

Table 10.—UBC seismic design parameters

Code	Z	K	C	Base shear V = ZKCW	Period (T)	J
					Second	
1967 UBC...........	1.00	0.67	0.057	0.038 W	0.70	0.64
1970 UBC...........	1.00	1.00	.057	.057 W	.70	.77

Maximum Story Shears

Maximum story shears were calculated as part of the dynamic analysis. Figures 25c and 26c indicate these as smooth curves with corresponding code values. Code design shears have been computed for the requirements of both the 1967 UBC, which was equivalent to the one used in the original design, and for the 1970 UBC, which would govern if the structure were to be built in 1971. The UBC story shears are the same for both directions. They are based on the numerical coefficients and periods shown in table 10.

The dynamic analysis determined peak values for the shears. These were calculated to have occurred at each floor level of the model at some time during the time-history response. These peak values did not necessarily occur simultaneously. They should be considered to represent only an envelope of maximum story shears.

The total response story shears were determined from the contribution of the first three modes of vibration, with the first-mode contribution accounting for much of the total response. Figures 25c and 26c also show the contributions of the second mode at the time of maximum story shears. The third mode-contributions were too small to show in the figures.

Maximum Overturning Moments

Envelopes of maximum overturning moments were calculated in a manner similar to that used to determine the maximum story shears. Figures 25d and 26d show these with corresponding 1967 and 1970 UBC values (except $J = 1.0$ was used for the 1970 UBC values). The first three modes of vibration were determined in the same way as the maximum story shears. Figures 25d and 26d also show the contributions of the significant modes to total response at the time of maximum response.

After publication of the 1970 UBC, the J factor for determining the base overturning moment received considerable scrutinization. Table 10 indicates that the 1970 UBC would require a minimum J value of 0.77, but subsequent amendments to the 1970 UBC have increased the minimum J value to 1.0. For this reason, overturning moments determined with a value of $J = 1.0$ have been used in figures 25d and 26d as a comparison with the 1967 UBC minimums in effect at the time the building was designed.

Loads on Key Structural Elements

Earthquake loads on structural elements in the lateral force-resisting frames were investigated by comparing seismic and estimated vertical load effects with the estimated capacity for several representative members. Hand analysis approximated the vertical dead and live load forces. The FRMDYN computer program produced the seismic forces.

As stated earlier, the models used in the FRMDYN computer runs were adjusted to respond at the measured periods. This was done by changing the modulus of elasticity, E, to change the stiffness.

In the shorter period models the value of E was increased. Apparently, part of this increase was used to offset the effects of the partitions. Also, part of the increase accounted for the nonlinearity of reinforced concrete, which has a higher E at lower amplitudes of structural strain.

The value of E was decreased in the longer period models. This change of E can be accounted for by a combination of nonlinearity at higher strains, loss of stiffness due to cracked sections, and inelastic behavior of some yielded elements. The values shown in tables 11 through 14 must be interpreted with the above remarks kept in mind.

Comparisons of girder loads, which include slabs and spandrels, were calculated using ratios of the controlling combinations of vertical load and seismic load moments to estimated ultimate capacities. Table 11 shows these as M/M_u for the transverse direction. Table 12 shows them for the longitudinal direction.

Ultimate moment capacities were computed by the recommendations of reference 3. Generally, girders were under-reinforced, indicating that the cross sectional area of available steel reinforcement, rather than crushing the concrete, limited ultimate moment capacity. In these calculations, the capacity reduction factor was $\phi = 1.0$.

Tables 11 and 12 divide the results of the girder investigation into two parts. The shorter period

Table 11.—Summary of girder $\frac{M}{M_u}$ for transverse frames

Frame	Floor	Reinforcement location	Model TD1 Exterior joint	Model TD1 1st interior joint	Model TD1 2d interior joint	Model TD2 Exterior joint	Model TD2 1st interior joint	Model TD2 2d interior joint
Transverse exterior frame	Roof	Top bars	1.9	1.1	1.4	2.9	1.6	2.0
		Bottom bars	0.6	0.3	0.9	1.4	0.9	1.9
	4th	Top bars	2.0	1.8	1.8	2.6	2.3	2.3
		Bottom bars	2.6	2.3	2.8	3.5	3.1	3.9
	2d	Top bars	1.8	1.7	1.6	2.6	2.4	2.1
		Bottom bars	2.3	1.8	2.0	3.6	2.9	3.0
Transverse interior frame	Roof	Top bars	1.0	.5	.5	1.3	.6	.6
		Bottom bars	.1			.4	.1	.1
	4th	Top bars	1.2	1.2	1.1	1.5	1.5	1.4
		Bottom bars	1.3	1.0	1.0	1.9	1.6	1.6
	2d	Top bars	1.1	.9	.8	1.5	1.2	1.1
		Bottom bars	1.3	.9	.8	2.0	1.6	1.6

Table 12.—Summary of girder $\frac{M}{M_u}$ for longitudinal frames

Frame	Floor	Reinforcement location	Model LD1 Exterior joint	Model LD1 1st interior joint	Model LD1 2d interior joint	Model LD2 Exterior joint	Model LD2 1st interior joint	Model LD2 2d interior joint
Longitudinal exterior frame	Roof	Top bars	0.9	0.6	0.7	1.0	0.6	0.8
		Bottom bars				.1		
	4th	Top bars	1.1	1.2	1.1	2.0	1.9	1.9
		Bottom bars	1.1	.7	1.2	2.3	1.8	3.1
	2d	Top bars	1.4	1.1	1.0	2.6	1.8	1.6
		Bottom bars	1.2	.7	.9	2.7	1.8	2.6
Longitudinal interior frame	Roof	Top bars	.8	.5	.5	.9	.5	.5
		Bottom bars						
	4th	Top bars	1.1	.7	.7	1.8	1.0	1.0
		Bottom bars	.3			.9	.7	.8
	2d	Top bars	1.0	.7	.7	1.7	1.0	1.1
		Bottom bars	.4	.2	.4	1.2	1.0	1.6

models, TD1 and LD1, represent the early portion of the motion. The longer period models, TD2 and LD2, represent the later portion of motion. For the shorter period models, a substantial number of girders have M/M_u ratios greater than 1.0, especially the exterior transverse frames, which have M/M_u ratios equal to or greater than 2.0.

Acknowledging the limitations of these results, there is still a reasonable indication that girders were beginning to yield after about 5 seconds of motion. For the longer period models, the M/M_u ratios are equal to or greater than 1.0 for almost all of the girders shown. Ratios greater than 3.0 were indicated for exterior transverse frames.

The ratio of M/M_u exceeding 1.0 implies that the reinforcing bars are yielding and redistribution of loads is occurring. Therefore, M/M_u values greater than 1 indicate fictitious moments and should be interpreted as such.

Comparisons of column loads were made by calculating ratios of the controlling combinations of vertical load and seismic load moments to estimated ultimate moment capacities computed on the basis of the recommendations of reference 3. The corresponding value for calculated axial load, P, is a ratio of P_{uo}. A capacity reduction factor of $\phi = 1.0$ was used for the rectangular tied columns.

Results of the column investigation in tables 13 and 14 are divided into two parts, shorter period models and longer period models. Table 13 gives the results for the transverse frames, and table 14 for the longitudinal frames.

The ratios of the shorter period models are all well below 1.0, except the corner columns in the transverse direction. About one-half of the moment ratios for the longer period models are equal to or greater than 1.0. The transverse corner columns have ratios greater than 2.0. Acknowledging the above limitations and the indications that girders yielded, there is doubt that the corner columns experienced some yielding.

Shear capacities of both girders and columns also

Table 13.—Summary of column interaction for transverse frames

Column and floor level	Model TD1		Model TD2	
	$\frac{M_x}{M_{ux}}+\frac{M_y}{M_{uy}}$	$\frac{P}{P_{uo}}$	$\frac{M_x}{M_{ux}}+\frac{M_y}{M_{uy}}$	$\frac{P}{P_{uo}}$
Typical exterior column:				
Roof	0.5	0.03	0.7	0.03
4th	.5	.1	.6	.1
2d	.5	.1	.6	.1
Typical interior column:				
Roof	.1	.06	.7	.06
4th	.5	.2	.7	.2
2d	.5	.2	.7	.2
Typical corner column:				
Roof	.9	.01	1.3	.01
4th	1.7	[1] .02	2.8	[1] .3
2d	1.5	[1] .08	2.2	[1] .3

[1] Column axial load in tension.

Table 14.—Summary of column interaction for longitudinal frames

Column and floor level	Model LD1		Model LD2	
	$\frac{M_x}{M_{ux}}+\frac{M_y}{M_{uy}}$	$\frac{P}{P_{uo}}$	$\frac{M_x}{M_{ux}}+\frac{M_y}{M_{uy}}$	$\frac{P}{P_{uo}}$
Typical exterior column:				
Roof	0.6	0.03	0.9	0.03
4th	.7	.2	1.4	.2
2d	.6	.2	1.2	.1
Typical interior column:				
Roof	.2	.06	.2	.06
4th	.2	.2	.5	.2
2d	.4	.2	.8	.2
Typical corner column:				
Roof	.7	.02	.7	.02
4th	.8	.08	1.4	.04
2d	.8	.05	1.4	.01

were checked under the requirements of reference 3. In general, ultimate girder shear capacities were not exceeded by combined vertical and seismic loads. Column shear stresses generally were less than ultimate capacities. However, the interior columns of the transverse exterior frames had calculated shears as high as twice the ultimate capacity. Again, we must acknowledge the limitations of the analysis and the redistribution of loads due to yielding.

DISCUSSION AND INTERPRETATION OF RESULTS

Comparison of Calculated Versus Code Forces

The previous paragraphs presented results of the dynamic analysis and subsequent comparisons of those results with code values. The results of the dynamic analysis, in general, showed that the level of code seismic forces was substantially less than what the structure was required to resist during the earthquake. Maximum base shears were calculated to be four to five times code requirements. Maximum overturning moments at the base of the structure were about nine times greater than code requirements in the transverse direction and about six times greater in the longitudinal direction.

Using the design seismic forces and fundamental period to compute displacement, the design code lateral roof displacement would be approximately 0.03 foot (roughly 0.07*g* at 0.70 second). This indicates that the peak lateral roof displacement was roughly 20 times the code value in the transverse direction and 16 times the code value in the longitudinal direction.

Examination of the computed forces in the moment-resisting frames indicated that a majority of the girders experienced excursions beyond their elastic bending moment yield capacities. Except for the exterior transverse frames, the columns remained within their elastic limit capacities. The columns in the exterior transverse frames were computed to have experienced high moment and shear forces. Because of the apparent yielding that occurred in the girders, these forces may have been redistributed. Uniformly distributed shear forces resulted in peak shear stresses of under 200 psi within the ultimate shear capacity of the reinforced concrete columns.

Modal Analysis Procedures

In order to test the accuracy of the mathematical models, an attempt was made to verify both mode shapes and periods. Mode shapes could be partially confirmed by the recorded motion by comparing fourth-floor and roof response data; but data would have to be obtained at additional floor-level locations to provide a sufficient number of data points to verify calculated mode shapes.

Period data played an important role in the comparison of the calculated periods with the recorded building periods. This comparison helped immensely to determine the participation of nonstructural elements and the effects of yielding. Since any one of several parameters could be changed to improve the correlation between calculated and measured periods, calculating the correct period did not insure absolutely that the mathematical model truly represented the actual structure. Verification of computed mode shapes by data obtained from building motion records at several intermediate levels would have improved the confidence level of the mathematical models.

Comparison of Recorded and Computed Responses

Comparing acceleration time histories for the roof and fourth-floor levels yielded comparisons of recorded and computed responses. The general shape of the computed time histories for particular time intervals correlates reasonably accurately with the recorded values (figs. 23 and 24).

Calculated fundamental periods for the structure (based on a bare structural frame) were approximately 0.79 second in the longitudinal direction and 0.88 second in the transverse direction. The apparent recorded fundamental period of the structure during the first 6 seconds of the earthquake was roughly 0.70 second in both the longitudinal and the transverse directions. The apparent recorded period of the structure during the latter portion of the earthquake was roughly 1.5 seconds in the longitudinal direction and 1.6 seconds in the transverse direction.

The shorter fundamental periods for the first few seconds of motion suggested that architectural elements initially provided the structure with additional horizontal rigidity. Guest rooms have a considerable number of party walls. There are exterior plaster walls and brick walls at the first floor. Although these walls are not considered part of the lateral force-resisting system, they do resist lateral forces, especially during the first part of the earthquake.

After 6 seconds of motion, the period of the structure appeared to lengthen in both directions. This indicated that after this time enough force had been generated to overcome most or all of the resistance of the nonstructural elements. In addition, the girders of the bare structural frames began to yield to further lengthen the periods.

Tables 8 and 9 list computed and recorded maximum accelerations and displacements. Comparisons between the computed and recorded results show reasonably good agreement.

Correlation With Damage Observations

The analyses have indicated that beams and slabs yielded, that columns generally remained within elastic limits (but the columns of transverse exterior frames were highly stressed), and that large interstory displacements, greater than 0.1 foot or 1 inch, occurred.

Observed damage included signs of yielding beams (figs. 9 and 10) and substantial nonstructural damage (figs. 11, 12, and 13). No serious column damage was observed, but cracked nonstructural architectural features were noted (fig. 14).

Beam damage appears less severe than one might imagine from a study of the results of the analysis. Yet these observations appear reasonable when compared to observations made on concrete test structures located at the Nevada Test Site (references 15 and 16). Observed partition damage also appears consistent when compared to published data (reference 11).

Ductility

The relatively large relative displacements exhibited by the analysis indicated a high degree of inherent ductility in the structure. A rough approximation of an average ductility ratio can be found by comparing the peak calculated fundamental mode relative lateral roof displacement with the equivalent displacement at the nominal elastic limit. The elastic limit displacement has been calculated to be 0.08 foot. This is roughly 2½ times the design code displacement.

Using the peak first-mode displacements shown in tables 8 and 9, the approximate ductility in the transverse direction would be 8. The approximate ductility in the longitudinal direction would be 6. The analysis of member forces indicates that this ductility was exhibited in the beams and slabs, while the columns essentially remain elastic.

Response Envelope Spectra

As previously indicated, the periods of response of the Holiday Inn structure lengthened during the earthquake. To give an illustrative representation of the change of period, spectral acceleration contours and spectral acceleration profiles were plotted for the transverse direction of the roof acceleration record (fig. 27).

The spectral acceleration contour mapping in figure 27a involved the plotting of contours of 5-percent damped spectral acceleration as a vertical dimension on a field plot of period versus elapsed time from the start of the recorded motion. The peak contours thus determined illustrate the period, amplitude, and duration of response of idealized simple oscillators located on the roof. They also show the relationships among various responses. In addition, the number of cycles of response motion above

any desired level can be estimated by dividing the duration of such motion by the period. Figure 27b shows vertical sections cut through figure 27a at the various response periods.

The response accelerations at the predominant building response periods are amplified to accentuate the time-history relationship of the predominant building period. The approximate time-history path of the fundamental period is illustrated by a dashed line in each figure. It indicates a fundamental period of 0.6 to 0.7 second for about 7 seconds, a transition until about 13 seconds, and then a fundamental period of 1.5 to 1.6 seconds.

Additional Period Data

In addition to the accelerograph recordings taken during the earthquake, building period observations were made before and after the earthquake. (See "Building Period Measurements Before, During, and After the San Fernando Earthquake.")

In the referenced observations, measured fundamental building periods (0.5 to 0.6 second) were significantly shorter than the periods observed during the earthquake; but postobservation periods were longer than preobservation periods. During these observations, the amplitude of motion was less than 1 percent of the recorded earthquake motion.

The results of these observations emphasize the greater participation of nonstructural elements at small amplitudes of motion. The relationship of the pre- and postearthquake observations to the earthquake observations agrees with similar types of observations in references 15 and 16.

SPECIAL STUDY ON VERTICAL RESPONSE

This section summarizes the results of a limited study of Holiday Inn (Orion Avenue) vertical response to the San Fernando earthquake.

The principal study objectives were: (1) to attempt to define the major parameters affecting the vertical seismic response of the building, and to estimate the proportionate significance of each; and (2) to evaluate the relative importance of vertical seismic forces in the overall building response to the February 9 earthquake.

Recorded Vertical Building Response

Figures 15, 16, and 17 furnish plots of the Holiday Inn (Orion Avenue) recorded strong-motion acceleration for the San Fernando earthquake. This section discusses data and interpretations relating only to the vertical component of earthquake motion and building response. Figures 28, 29, and 30 present response spectra prepared from the digitized time-history record of vertical accelerations measured at the first floor, fourth floor, and roof.

Reviewing the digital acceleration time-history data, table 2 shows the peak vertical accelerations and times of occurrence from the beginning of the record. They were 0.18g at 3.6 seconds on the first floor, 0.24g at 3.5 seconds on the fourth floor, and 0.24g at 3.6 seconds on the roof. The relative locations of the instruments somewhat complicate direct comparison of data (figs. 4, 5, and 6). The periodicity of the first-floor motion and the effects of two damping values are displayed in figure 28. The predominant period of vertical earthquake motion is about 0.3 second. The major unit spectral energy lies in the period range of about 0.2 to 0.4 second.

To distinguish the predominant vertical response of the building from the ground response, represented by the first-floor data, the first-floor spectrum was plotted with the fourth-floor and roof spectra. Notice the regions near the periods of 0.12 and 0.21 second in figures 29 and 30. At the fourth-floor and roof level, amplifications of the first-floor spectral accelerations attract attention, particularly near the period of 0.12 second. By careful overlay of the first- and fourth-floor vertical acceleration time-history traces (figs. 16 and 17), a response at the fourth floor at a period of about 0.12 second can be distinguished, but the occurrence of a response at 0.21 second is not clearly defined.

From these data, the predominant vertical building response is interpreted to be near the period of 0.12 second. Further electronic processing and filtering of available records could furnish verification of the predominant vertical building response characteristics, and possibly some insight to the apparent amplified motion near the period of 0.21 second.

Mathematical Model for Vertical Response

The Holiday Inn vertical response analysis was limited to a first approximation study of a two-dimensional vertical element of the building, consisting of a vertical line of interior columns and the effective tributary flat slabs at each level. The vertical element with column axes at the intersection of building lines 5 and C was selected since this loca-

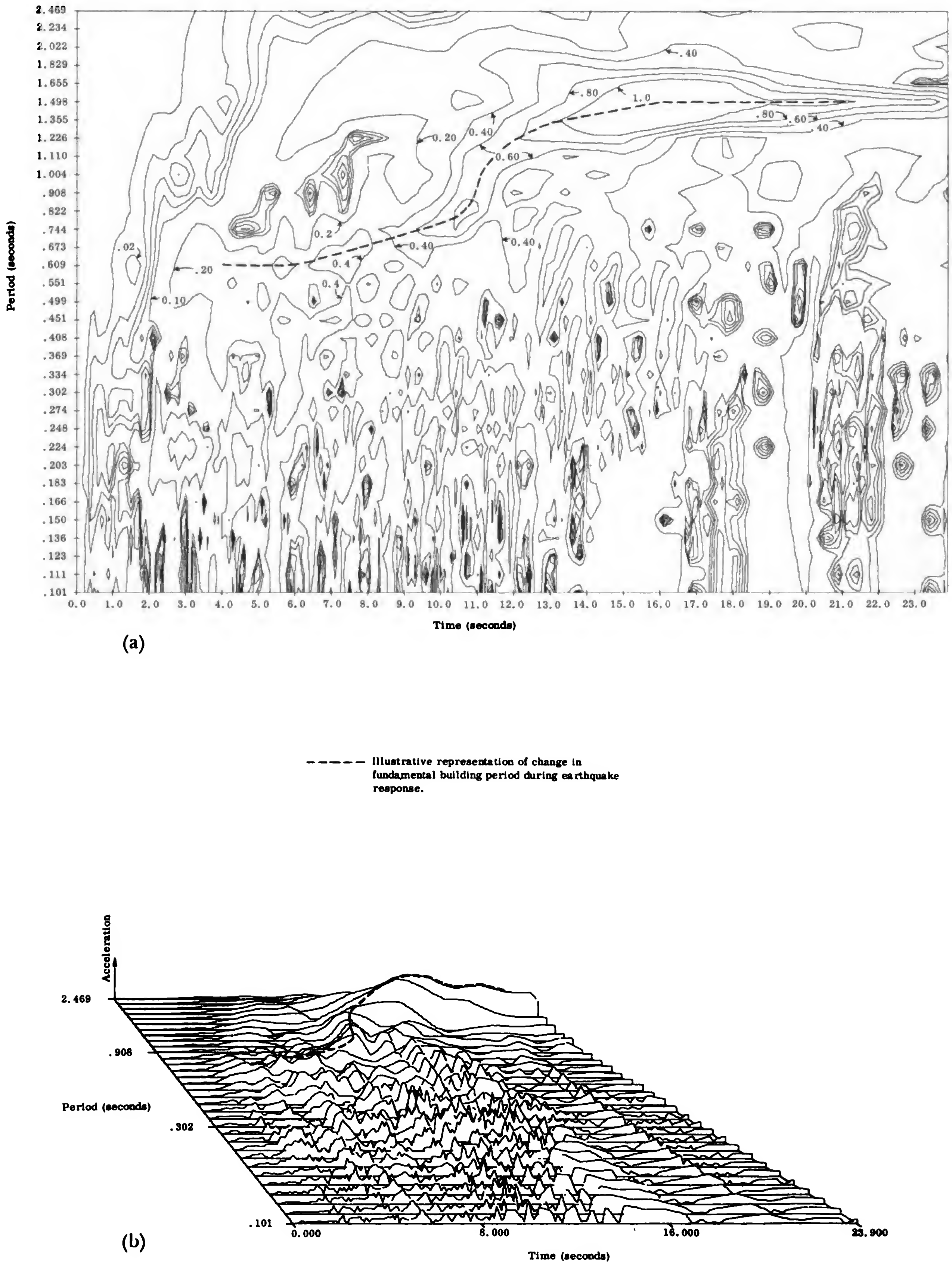


Figure 27.—Holiday Inn, Orion Avenue. Response envelope spectra (5-percent damped) using transverse direction of roof acceleration record. (a) Special acceleration contours (Contours represent accelerations in g). (b) Spectral acceleration profiles.

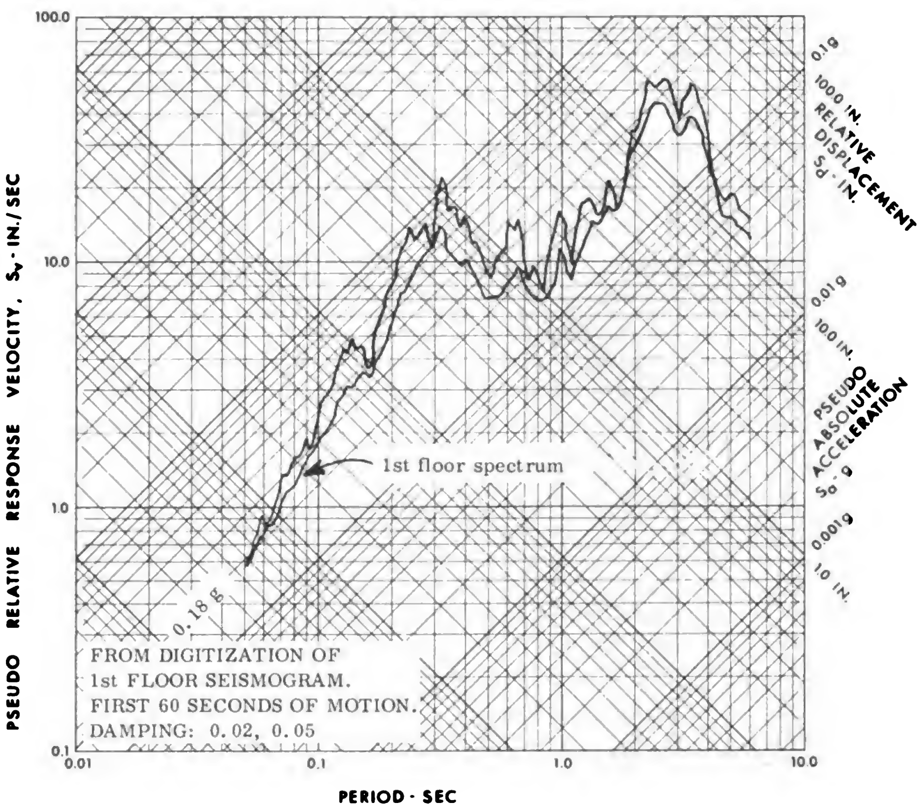


Figure 28.—Holiday Inn, Orion Avenue. Vertical response spectra at first-floor level.

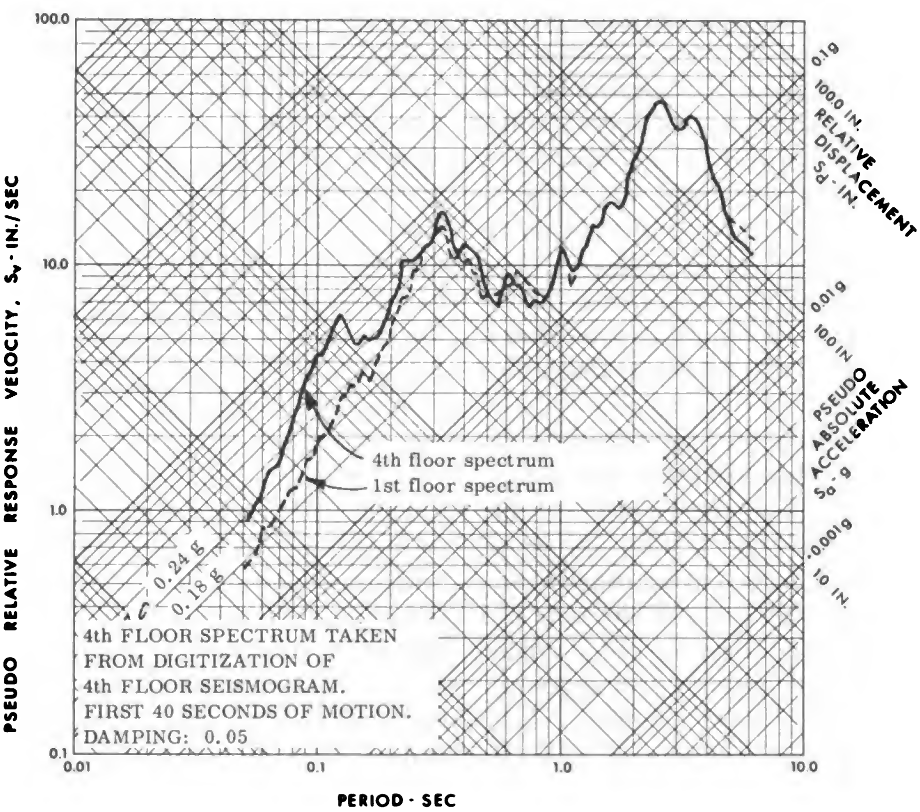


Figure 29.—Holiday Inn, Orion Avenue. Comparison of vertical response spectra at first- and fourth-floor levels.

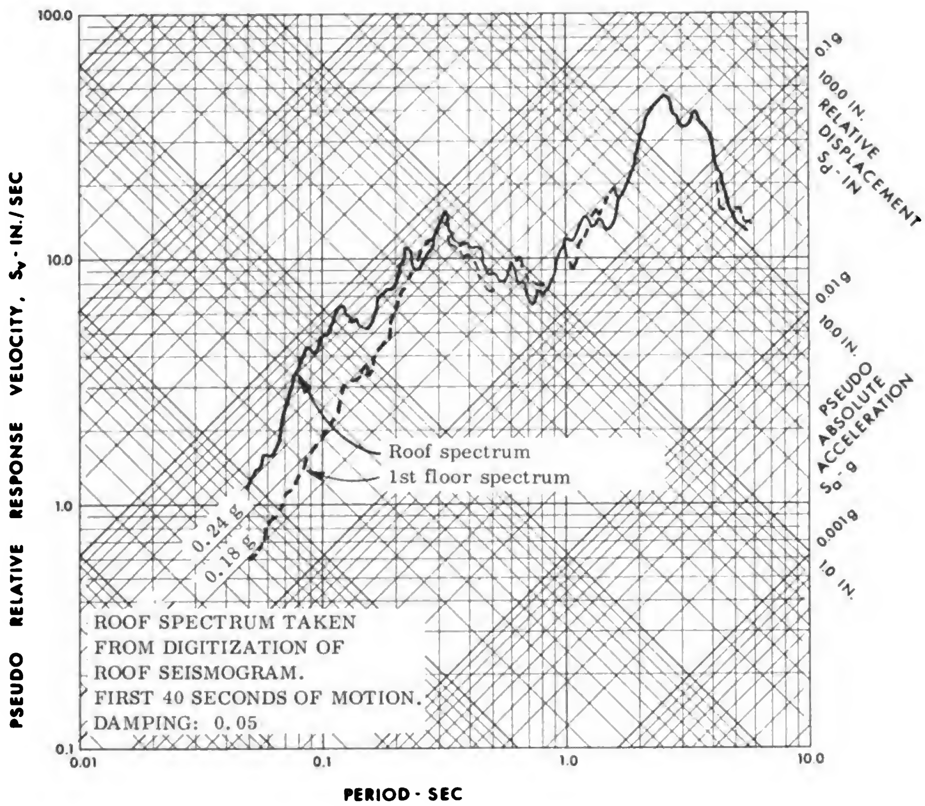


Figure 30.—Holiday Inn, Orion Avenue. Comparison of vertical response spectra at first-floor and roof levels.

tion lies nearest the strong-motion recorder on the fourth floor.

The above structural element was idealized as an elastic, vertical, lumped-mass system with spring-supported column masses lumped at each floor level. The effective tributary slab masses are considered to be spring-supported from rigid horizontal links attached to the column masses (fig. 31, model C). The horizontal link is used only as a convenience in describing the coupling of the slab mass to the column mass at each level. It does not represent any other property of the structure.

Since the recorded acceleration time history at the first-floor level is used as input motion to the model, the foundation system is not represented in the model. The model designates two vertical degrees of freedom at each floor level, representing the predominant vertical translation mode only of the slab and column elements. The flat-slab element predominant mode shape assumed in this analysis, and described later, does not result in net column-bending actions. Refinements considering higher modes of slab element bending, column element rotational degrees of freedom, etc., require a more sophisticated analytical approach beyond the intended scope of this study.

Note that the vertical model corresponding to the roof-level strong-motion recorder location would be different from this study subject model of a typical interior region of the building. The roof-level recorder is located on the southwest stairwell 6-inch-thick two-way slab (fig. 6). The effective tributary slab and supporting column characteristics at this location, from the ground level upward, would bear little or no relationship to the properties at the building interior.

For correlation of measured and calculated fourth-floor level response accelerations, the simplifying assumption that vertical motion can be considered decoupled from the horizontal is justifiable. Vertical components of acceleration due to combined column shortening and slab flexure caused by the horizontal motion are comparatively small. They occur in a much longer period range than the predominant vertical response. However, variations in boundary conditions for the slab element, due to horizontal seismic effects, are considered in the estimate of slab stiffness properties.

Generally, to evaluate the effects on vertical response of various assumptions for the model parameters, several different mathematical models were

considered. The following paragraphs discuss the range of model properties in the three general categories of effective slab mass and stiffness, column mass and stiffness, and damping. Such quantities as the unit weight and modulus of elasticity of concrete, modular ratio between steel and concrete, etc., are held constant throughout the analyses since there were not any measured data available to judge their variation. The effect of variations in these quantities can be judged generally from the results of changes in effective stiffness in the model.

The single-degree-of-freedom system representing the flat-slab dynamic properties is probably the most uncertain element of the vertical model. Considered as a whole, the major portion of the building has 24 flat-slab elements supported by 36 columns per floor. By excluding the exterior columns and one-half of the perimeter slabs (to represent the interior portion of the building), a ratio of one column per slab results. Since a representative interior region of the building is modeled, the effective support is considered to be a single column per slab.

The equivalent single-degree-of-freedom slab element in the model is obtained from the data for the actual flat slab by equating the total work done and the kinetic energy of the equivalent system, up to any time of response history, to that of the actual slab for an assumed predominant, elastic, deformed shape of the slab. Since only one vibration mode is represented in the equivalent single-degree-of-freedom slab element, the slab-deflected shape should correspond to the lowest mode that can be excited by the vertical seismic motion and is nearest the predominant period of the motion.

For the model, the slab-deflected shape at each level was taken as that of a uniformly loaded square interior panel (reference 13). Calculations show that corresponding periods for the slabs at each level are typically in the range of one-half the predominant period of the ground motion vertical component. Higher slab modes would be of relatively less significance (fig. 28).

The single-degree-of-freedom slab element, as described, should then reasonably approximate the dynamic characteristics of an actual interior flat slab. The effective slab mass becomes about 34 percent of the mass calculated for the actual slab, and the effective stiffness is about 53 percent of the calculated actual slab stiffness. The slab element mass point effective accelerations in the model are referred to as the midpoint of the flat slab. However, since a unique predominant deflected shape is used to describe the single-degree-of-freedom slab element, the accelerations in the model at the fourth-floor instrument location can be determined readily.

The flat-slab reinforcing steel pattern gives transformed moments of inertia that differ in each direction, as well as along the column and midstrips. Also, the horizontal bending deformations superimposed on the vertical plate type bending shape

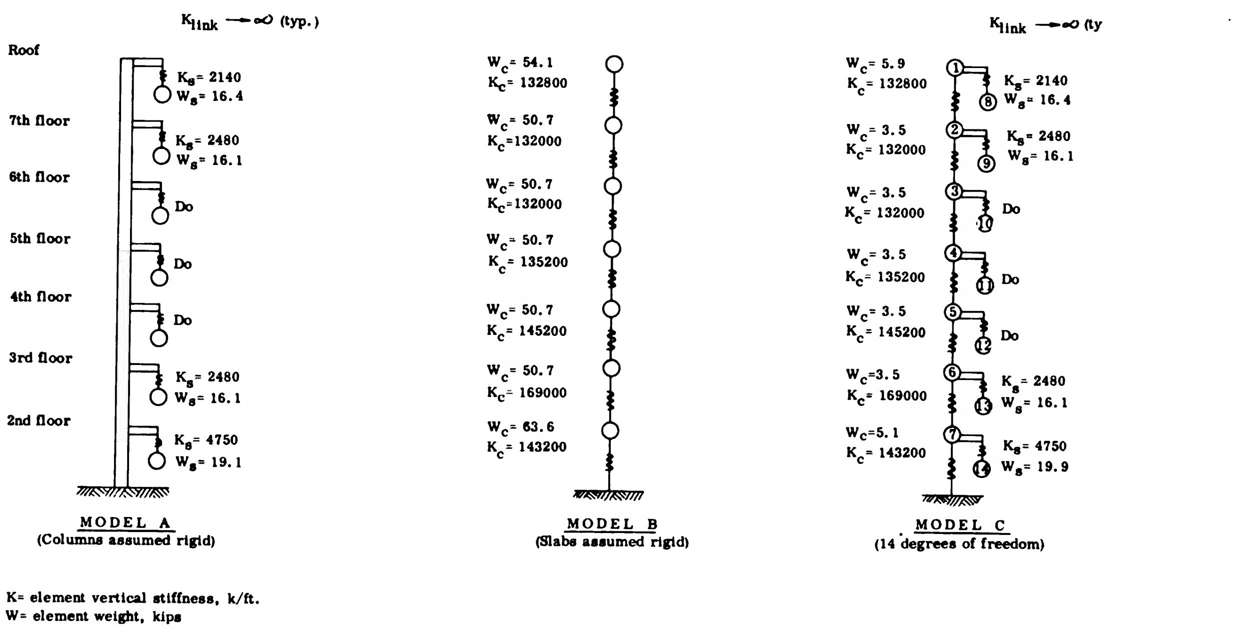


Figure 31.—Holiday Inn, Orion Avenue. Vertical response mathematical models.

throughout the time histories of response result in variations between cracked and gross section properties.

An average of the theoretical cracked section and gross section properties over the slab in both directions is considered to be a reasonable approximation of the slab effective moment of inertia for the vertical response study. The slab weight is taken as the sum of the structural weight and the tributary architectural weight of partitions, fixtures, suspended ceiling, etc. Both the calculated mass and stiffness are modified as described to arrive at the appropriate slab element dynamic characteristics.

The column mass properties in the model at each level are derived from the tributary column structural and architectural weight and the portion of slab weight included within the column punching shear region. Considering the dead load on the column, the vertical spring constant is calculated as the uncracked axial stiffness of the column, including the transformed area of vertical reinforcement. For this building, the ratio of slab weight to supporting column weight is about 14 to 1. The ratio of column axial stiffness to slab effective vertical stiffness exceeds 50 to 1.

The overall effect on the model of variations in column mass and stiffness properties is then practically negligible, and may not be of concern once the appropriate range of values is determined. As previously noted, direct comparisons of recorded and calculated absolute accelerations at the roof level are not consistent theoretically. However, for rough comparisons, the calculated accelerations at the roof column mass point were selected as the closest representative values.

For a particular degree of freedom, it is commonly considered that damping has some functional relationship to the corresponding intensity of motion and stress levels introduced within the structure, and character of nonstructural energy-absorbing mechanisms present, such as partitions, suspended ceilings, etc. (references 14, 15, and 16). Vertical motion considered independently, in terms of maximum displacements calculated from instrumental data, would suggest a relatively low value of damping, probably around 2 percent of critical. However, for some portion of the vertical response time history where slab cracking occurs due to horizontal seismic motion, vertical damping is more likely to be a larger ratio such as 5 percent or more.

In the period range of 0.10 to 0.15 second, figure 28 shows that a change in damping ratio from 5 to 2 percent can result in a change in spectral acceleration of from about 10 to 50 percent. Therefore, the determination of modal damping ratios is a relatively important factor.

For this limited study, the characteristics briefly discussed above were considered as the major variables affecting the vertical seismic response of Holiday Inn, Orion Avenue.

Analytical Procedures for Vertical Response

To review the performance of the mathematical model, approximate analyses for hypothetical extremes of slab and column stiffnesses were first computed by hand, using response spectrum methods. For all hand calculations, the Rayleigh method, where all masses were assumed to be displaced in the same direction in the model fundamental mode, was used.

The model reduced to a mutually decoupled, 7°-of-freedom slab element system when the columns are assumed rigid relative to the slabs (fig. 31, model A). For the alternate extreme of the slabs assumed rigid relative to the columns, a coupled 7°-of-freedom column system results (fig. 31, model B). Because of the coupling in the real system, these two extremes do not necessarily bound the vertical response for a given damping ratio. A 14°-of-freedom model also was analyzed by hand, using calculated representative values of slab and column element effective mass and stiffness (fig. 31, model C).

The hand-calculated vertical periods and peak absolute accelerations were compared with recorded data. Since the strong-motion recorders measure absolute accelerations, it is necessary to add a ground acceleration component to the effective accelerations calculated from the model to make comparisons. For hand calculations, the added ground component was taken by an accepted method as the peak vertical first-floor (considered ground) acceleration multiplied by a factor varying from 1.0 at the base to 0 at the roof level.

Based on hand-calculated results, it was determined that the 14°-of-freedom mathematical model (fig. 31, model C) generally would be appropriate for the scope of vertical response review. Further analysis was performed using the MATRAN computer program. In order to evaluate estimated model parameters, such as damping, correlation studies

were made between computed and measured periods and response accelerations at the fourth-floor level. Computed maximum dynamic flat-slab bending moments and column forces were compared with code dead and live load design values.

Results of Vertical Analyses

Mathematical models used in the hand-calculated analyses are shown in figure 31. Table 15 shows the results. Interestingly, all three models produce nearly the same fundamental period close to the predominant period of about 0.12 second determined from instrumental records. The rigid column, model A, and 14°-of-freedom model C yield comparable results of fourth-floor accelerations in reasonable agreement with measurements. The rigid slab model B overestimates response accelerations.

For the assumption of rigid slabs, the mass lumped at any level becomes the sum of the calculated actual column and slab masses, which is more than twice the effective mass with slab flexibility considered. The 14°-of-freedom model was considered to give reasonably approximate results. More detailed analyses were performed using the MATRAN program with damping as the primary variable.

Table 15.—Hand-calculated results for vertical response

	Fundamental period	4th-floor maximum absolute acceleration		Roof maximum absolute acceleration [1]	
		$\lambda = 2$ percent	$\lambda = 5$ percent	$\lambda = 2$ percent	$\lambda = 5$ percent
	Second	*g*	*g*	*g*	*g*
Model A.......	0.09	[2] 0.23	[2] 0.20	0.18	0.18
Model B.......	.10	.41	.34	.50	.40
Model C.......	.11	[2] .26	[2] .22	.20	.15
Recorded......	.12	0.24		0.24	

[1] Absolute accelerations at roof column location.
[2] Absolute accelerations at instrument location.

Tables 16, 17, and 18 show the results obtained by use of the MATRAN program for model C of figure 31. Computations demonstrated that if only the first seven modes of the 14°-of-freedom system are considered, about 10 percent of the total response of the model is truncated.

Table 16 presents the general response characteristics of the model. Note the close grouping of periods for the first seven modes representing primarily slab response as seen from the mode shape tabulation. The seventh mode is essentially all second-floor slab response. The eighth and higher modes are predominantly the actions of the columns. Effective modal loads are relatively negligible beyond the ninth mode. Table 17 shows maximum computed vertical response and comparisons of maximum dynamic response to code dead and live load design values. Table 18 compares the recorded peak accelerations with calculated peak accelerations adjusted for instrument locations.

Discussion and Interpretation of Results of Vertical Study

It was determined that a simplified vertical model of the typical interior building region considered decoupled from the horizontal structure characteristics

Table 16.—Computed vertical dynamic properties

Mode number.	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Period (second).	0.109	0.095	0.091	0.090	0.090	0.089	0.073	0.024	0.010	0.006	0.004	0.004	0.003	0.003
Mass point [1]	Mode shapes (normalized ratios)													
1	0.205	−0.038	−0.021	−0.007	0.003	0.001	−0.018	−1.488	1.193	−0.938	−0.723	−0.516	0.318	0.091
2	.192	−.054	−.024	−.008	.003	.001	−.019	−1.372	.509	.435	1.242	1.669	−1.494	−5.18
3	.170	−.062	−.012	.004	−.006	−.003	−.020	−1.206	−.349	1.440	1.215	−.334	1.744	.942
4	.141	−.062	.008	.010	.001	.004	−.023	−.996	−1.091	1.200	−.772	−1.496	−.927	−1.364
5	.108	−.053	.022	−.000	.005	−.004	−.026	−.756	−1.460	−.047	−1.497	1.046	−.469	1.710
6	.072	−.038	.023	−.009	−.004	.002	−.030	−.508	−1.362	−1.171	.024	1.019	1.401	−1.645
7	.040	−.021	.013	−.005	−.002	.001	−.035	−.280	−.923	−1.346	1.300	−1.008	−.700	.610
8	.992	.972	.146	.044	−.018	−.006	.023	.097	−.012	.004	.002	.001	−.000	−.000
9	.584	−.446	−.796	−.706	.511	.230	.037	.107	−.006	−.002	−.003	−.003	.002	.001
10	.518	−.517	−.391	.372	−.887	−.612	.040	.094	.004	−.007	−.003	.001	−.002	−.001
11	.431	−.516	.255	.846	.225	.842	.045	.078	.013	−.006	.002	.002	.001	.001
12	.329	−.443	.732	−.032	.689	−.823	.051	.059	.017	.000	.004	−.002	.001	−.002
13	.221	−.318	.767	−.802	−.652	.434	.060	.040	.016	.005	−.000	−.002	−.002	.002
14	.070	−.049	.034	−.014	−.007	.003	−1.268	.035	.017	.010	−.005	.003	.001	−.001
	Effective modal loads (ratio to total weight)													
	.647	.107	.032	.005	.001	.000	.102	.075	.021	.007	.002	.000	.000	.000

[1] For location of mass points, see figure 31, model C.

Table 17.—Maximum computed vertical response

Location	Maximum absolute acceleration		Maximum absolute displacement		Maximum dynamic force, P_{DYN}		$\frac{P_{DYN}}{P_{D+L}}$ or $\frac{(M_o)_{DYN}}{(M_o)_{D+L}}$ [1]	
	$\lambda = 2$ percent	$\lambda = 5$ percent	$\lambda = 2$ percent	$\lambda = 5$ percent	$\lambda = 2$ percent	$\lambda = 5$ percent	$\lambda = 2$ percent	$\lambda = 5$ percent
	g	*g*	*Inch*	*Inch*	*kips*	*kips*		
Roof slab..................	0.77	0.45	0.090	0.050	12.5	7.3	0.24	0.14
4th-floor slab..............	.33	.29	.036	.031	5.3	4.7	.09	.08
2d-floor slab...............	.30	.24	.016	.014	5.9	4.7	.09	.07
Roof column...............	.23	.21	.020	.014	13.8	8.3	.26	.16
4th-floor column............	.20	.19	.011	.008	42.0	29.2	.14	.10
2d-floor column............	.18	.18	.004	.003	48.5	38.6	.11	.09

[1] Ratio of calculated vertical response force P_{DYN} or moment $(M_o)_{DYN}$ to calculated dead plus live load force P_{D+L} or moment $(M_o)_{D+L}$.

Table 18.—Comparison of recorded peak accelerations and computed peak accelerations adjusted for instrument location

	Maximum absolute accelerations at 4th-floor instrument location		Maximum absolute accelerations at roof instrument location	
	$\lambda = 2$ percent	$\lambda = 5$ percent	$\lambda = 2$ percent	$\lambda = 5$ percent
	g	*g*	*g*	*g*
Computed from model............	0.25	0.23	[1] 0.23	[1] 0.21
Determined from records...........	0.24		0.24	

[1] At column location.

can produce analytical results in reasonable agreement with the corresponding recorded earthquake data (table 18). In predictions of vertical response, the action of the more massive and less stiff floor system is relatively more significant, particularly in the lower modes, than that of the lighter and stiffer columns.

However, the higher mode column action does somewhat amplify the overall building vertical response at any level. Although nothing conclusive regarding damping was demonstrated in this study, damping in the vertical direction is likely to be influenced by the strain ranges in the structure resulting from the occurrence of simultaneous horizontal motion.

As a result of this study, the maximum vertical seismic response is on the order of 20 percent of the dead plus live load moments for the flat slabs, and 20 percent of the dead plus live axial loads for the columns (table 17). The vertical seismic flat-slab maximum bending moments seem small relative to those caused by the horizontal response. For the interior columns, although the horizontal seismic axial loads are small, interacting column bending moments are very large, overshadowing the relative importance of the vertical response.

The primary concern of vertical seismic actions may be in corner columns under uplift conditions where the addition of vertical loads acting upward on columns already in the tension region of interaction may be significant. However, further detailed studies should be made considering coupled vertical and horizontal seismic response.

SUMMARY OF FINDINGS

The dynamic analysis and special study of vertical response of the Holiday Inn on Orion Avenue yielded the following conclusions about the effects of the San Fernando earthquake.

1 During the earthquake, the structure responded at amplitudes that exceeded the elastic limits of a substantial number of girders.

2 Calculated earthquake forces exceeded prescribed code minimums by a factor of 4 or 5.

3 Interstory displacements exceeded 1 inch. This accounts for the large amount of nonstructural damage.

4 On the basis of calculated building response, maximum lateral displacements were from 16 to 20 times prescribed code minimums, but observed structural damage was relatively minor.

5 Estimates of ductility indicate average ratios of 8 for beams and slabs in the transverse direction and 6 for beams and slabs in the longitudinal direction.

6 Damping was approximated to be 5 to 10 percent of critical viscous damping.

7 Although there was some vertical amplification of vertical ground motion, peak resulting stresses were only approximately 20 percent of dead and live load stresses.

RECOMMENDATIONS

This study recommends several areas of possible change in seismic design and instrumentation.

Dynamic Analysis

Presently, the state-of-the-art of structural engineering includes some very useful methods for the dynamic analysis of structures. These include time-history analysis, response spectra model analysis, and computerized analytical methods of structural analysis. With the growing use of these complex analytical methods, design codes should establish criteria to govern the use of these procedures. The results of the analysis of the Holiday Inn structure have illustrated the variation of results that can be obtained because of changes in natural periods of vibration. It should be noted that none of these methods provide exact solutions to dynamic response. Considering their limitations, they yield reasonable representations. Some of the limitations include instrument accuracy, time scales, phase relationships between modes in both recorded and calculated time histories, random variables, human error, and effects of instrument locations.

Modal Periods

These reports illustrate the nonlinearity of structural response and its effects on modal periods. Similar findings have been observed in other investigations. Most dynamic analysis procedures consider each period to have a constant value. Yet it appears that structures should be analyzed for a spectrum of periods. The value for each period would be dependent on amplitude of building motion. Design criteria should be established to incorporate these effects of potential period variation.

Nonstructural Elements

As has been illustrated in these reports, the partitions, as well as other nonstructural elements, can play an appreciable role in the character of the structural response of a building to an earthquake. The effects of nonstructural elements can add to or subtract from the lateral capacity of structures. Therefore, it is recommended that the effect of nonstructural elements be included in the lateral force design criteria.

Strong-Motion Instrumentation

Strong-motion recording devices should be placed to provide a record of true transverse and longitudinal motion without any undue contributions from real or accidental torsional effects. Vertical records should be taken at or near a column to avoid local, possibly anomalous, effects from more flexible elements, such as thin slabs. It also may be desirable to locate some vertical recorders at the center of typical floor elements to measure possible response amplification.

Verification of mode shapes could be improved if a greater number of recording instruments were placed between the top and ground levels of high-rise buildings. By placing only one intermediate or midheight instrument, verification may be doubtful.

Vertical Response

The results of the study, although limited, indicate that vertical response may not be a critical factor in the dynamic stress analysis of a building-type structure. However, the analysis did indicate that amplification of the vertical response can occur. This may affect objects attached to or placed in the building.

Holiday Inn (30)

1640 Marengo Street, Los Angeles

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JOHN A. BLUME & ASSOCIATES, ENGINEERS
San Francisco, Calif.

DESCRIPTION OF BUILDING

The Holiday Inn is a seven-story reinforced concrete structure. Typical plan dimensions are about 62 by 160 feet (fig. 1). Located adjacent to the intersection of the Golden State Freeway and the San Bernardino Freeway, near the Boyle Heights section of Los Angeles, the structure is about 26 miles south of the epicenter of the San Fernando earthquake. This area is on the northeast side of the Los Angeles Basin. Geological source data indicate that the site lies on older alluvium. The typical soil boring log (fig. 2, reference 17) shows the underlying soil to be primarily fine sandy silt, clayey silt, and silty fine sand.

The structure, which consists of roughly 63,000 square feet of floor space, was designed in 1965. Constructed in 1966 at a cost of approximately $1.3 million, this building is essentially identical to the Holiday Inn located at 8244 Orion Avenue (covered in the preceding report). Some comparisons between the two buildings will be made in this building report.

The foundation system (fig. 3) consists of 36-inch-deep pile caps, supported by groups of two to four poured-in-place 24-inch-diameter reinforced concrete friction piles centered under the main building columns. A grid of tie beams and foundation beams connects all pile caps. Each pile is roughly 35 feet long. Each has a design capacity of over 100 kips vertical load and up to 20 kips lateral load (reference 17).

The first floor is a slab on grade over about 2 feet of compacted fill. Except for two small areas at the ground floor, which are covered by one-story canopies, the plan configurations of each floor, including the roof, are the same (figs. 4, 5, and 6). The typical framing consists of columns spaced at 20-foot centers in the transverse direction and 19-foot centers in the longitudinal direction. Spandrel beams surround the

Figure 1.—Holiday Inn, Marengo Street. Northwest elevation. John A. Blume & Associates photograph.

perimeter of the structure. The floor system is a reinforced concrete flat slab, 10 inches thick at the second floor, 8½ inches thick at the third to seventh floors, and 8 inches thick at the roof (fig. 7). A penthouse with mechanical equipment covers approximately 10 percent of the roof area.

The structure is constructed of regular weight reinforced concrete. Table 1 gives the properties of the structural materials specified for the construction.

Interior partitions, in general, are gypsum wallboard on metal studs. Cement plaster, 1 inch thick, is used for exterior facing at each end of the building and at the stair and elevator bays on the long side of the building. Double 16-gauge metal studs support the cement plaster.

Some additional cement plaster walls are located on the south side of the building at the first floor. The north side of the building, along column line D, has four bays of brick masonry walls located between the ground and the second floor at the east end of the structure. Nominal 1-inch expansion joints separate these walls from the exterior columns. Nominal ½-inch expansion joints separate the walls from the underside of the second-floor spandrels. Although none of the wall elements described are designed as part of the lateral force-resisting system, they do contribute in varying degrees to the stiffness of the structure.

Lateral forces in each direction are resisted by the interior column-slab frames and by the exterior column-spandrel beam frames. The added stiffness af-

Table 1.—Properties of construction materials

Concrete (regular weight, 150 pcf [1] unit weight)

Location in structure	Minimum specified compressive strength (f'_c)	Modulus of elasticity (E)
	psi [2]	*psi* [2]
Columns, 1st to 2d floors.............	5,000	4.2×10^6
Columns, 2d to 3d floors.............	4,000	3.7×10^6
Beams and slabs, 2d floor.............	4,000	3.7×10^6
All other concrete, 3d floor to roof.....	3,000	3.3×10^6

Reinforcing steel

Location in structure	Grade	Minimum specified yield strength (f_y)	Modulus of elasticity (E)
		ksi [3]	*psi* [2]
Beams and slabs.....	Intermediate-grade deformed billet bars (ASTM A-15 and A-305).	40	29×10^6
Column bars.......	Deformed billet bars (ASTM A-432).	60	29×10^6

[1] Pounds per cubic foot.
[2] Pounds per square inch.
[3] Kips per square inch.

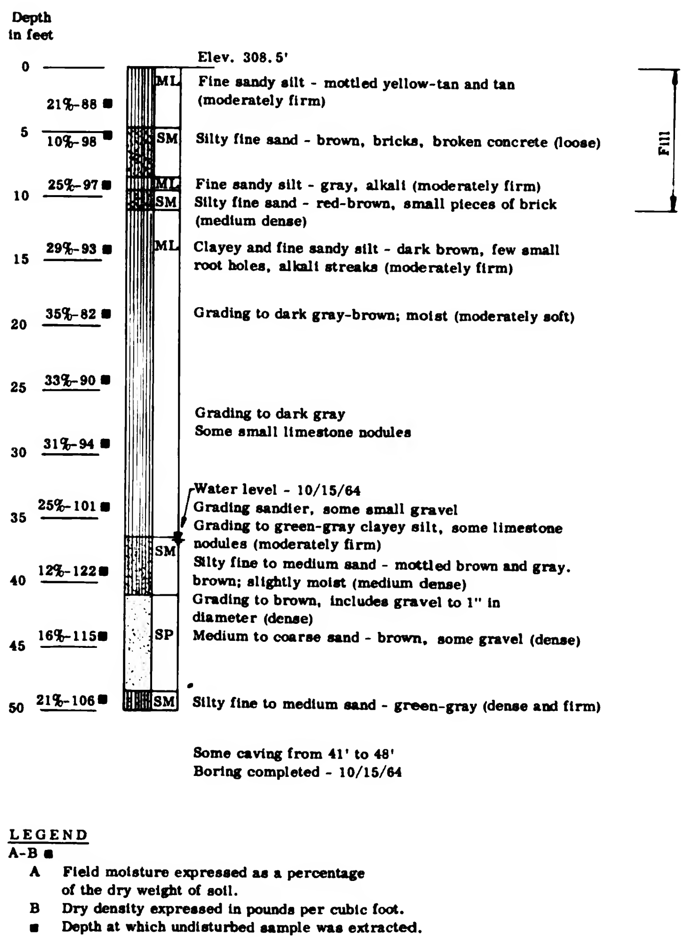

Figure 2.—Holiday Inn, Marengo Street. Log of typical soil boring.

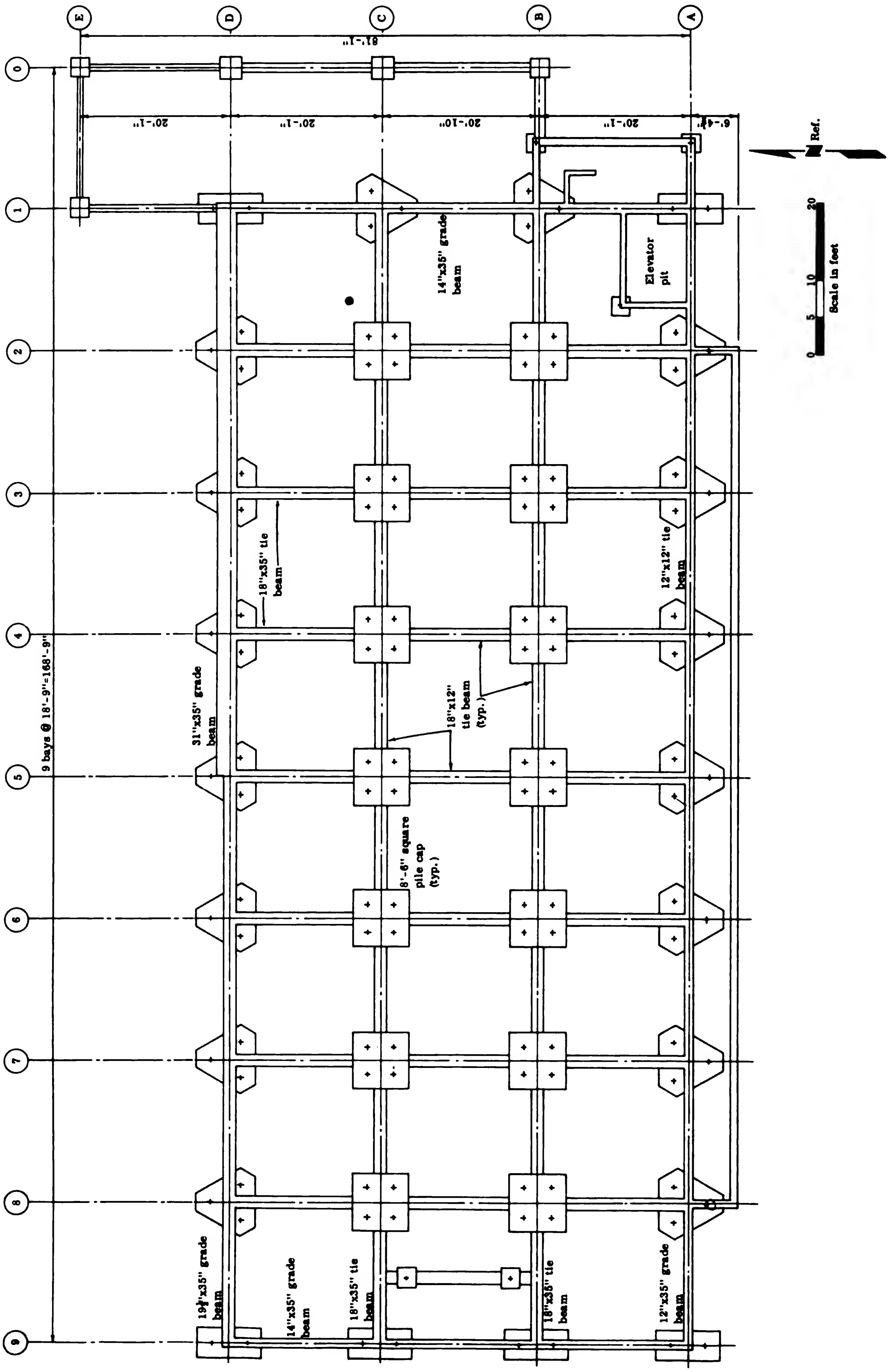

Figure 3.—Holiday Inn, Marengo Street. Foundation plan.

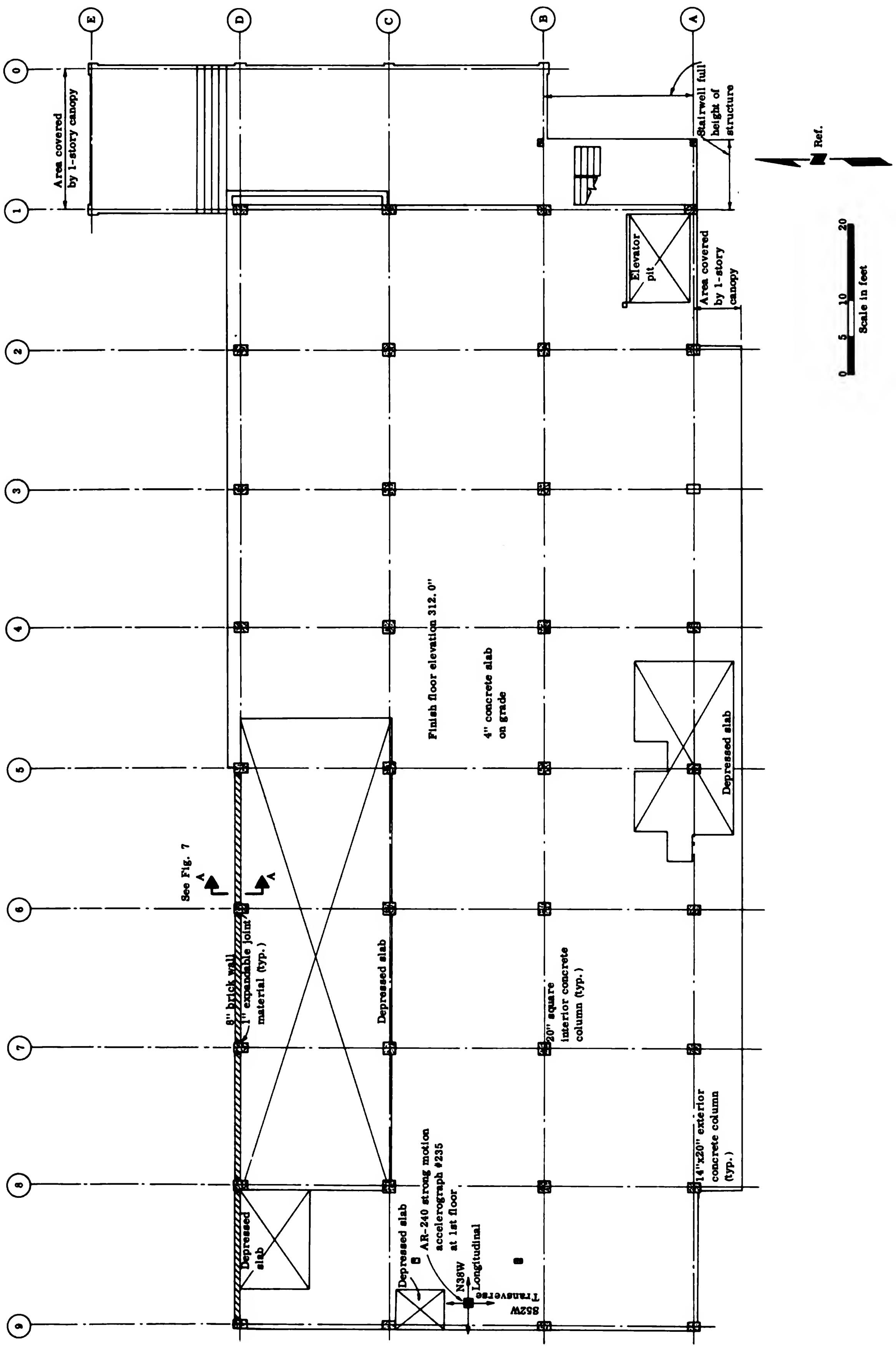

Figure 4.—Holiday Inn, Marengo Street. First-floor plan.

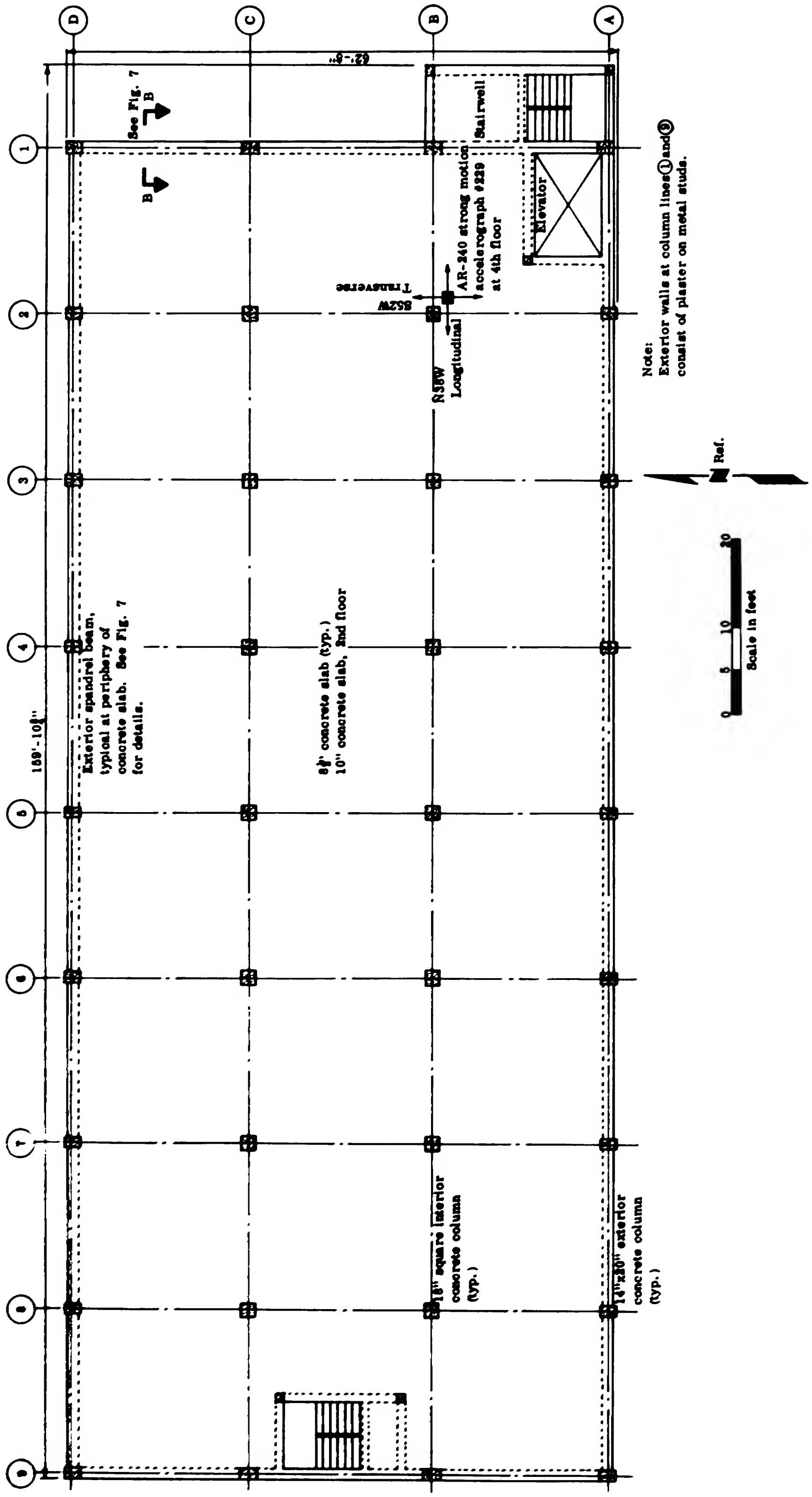

Figure 5.—Holiday Inn, Marengo Street. Typical floor framing plan.

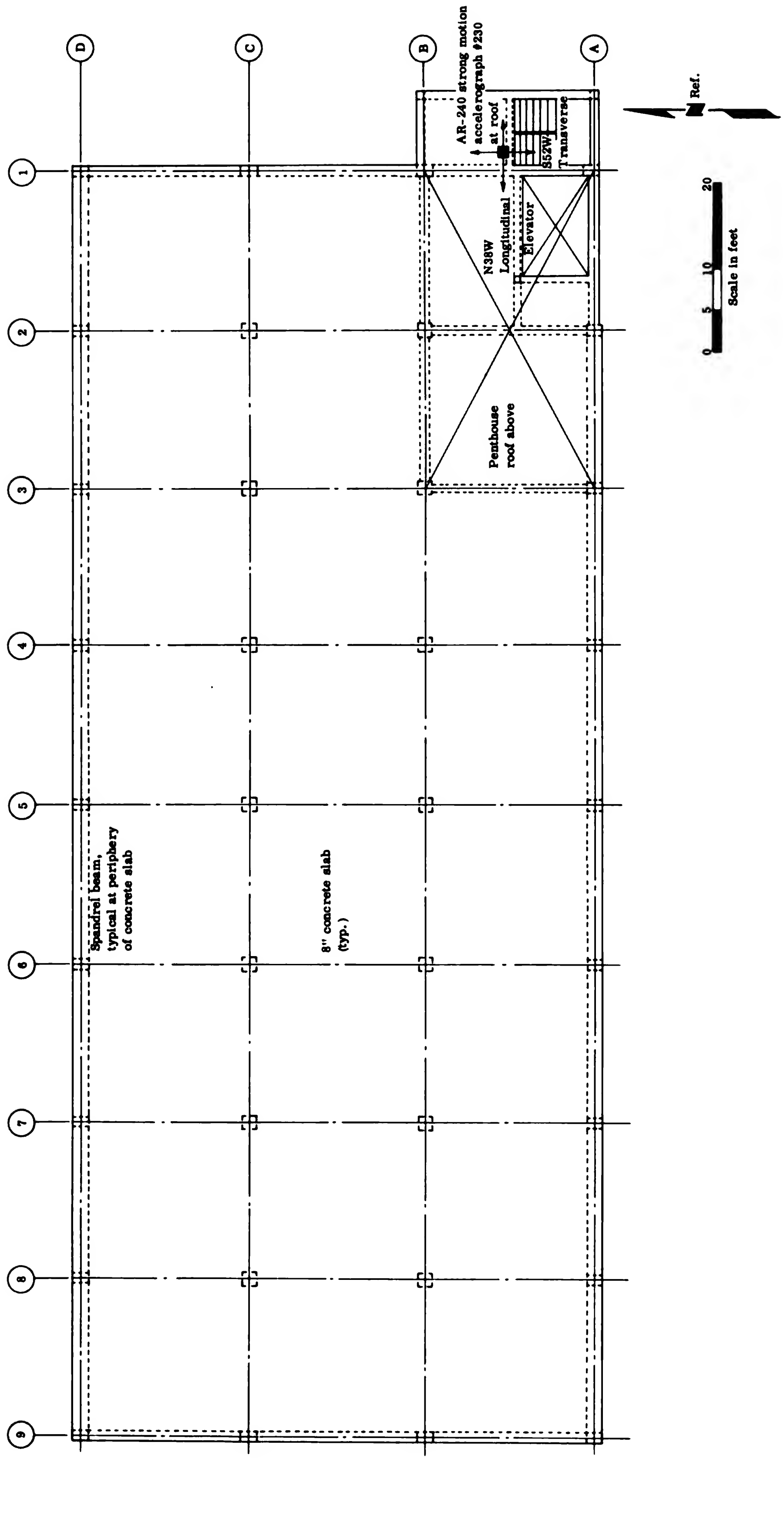

Figure 6.—Holiday Inn, Marengo Street. Roof framing plan.

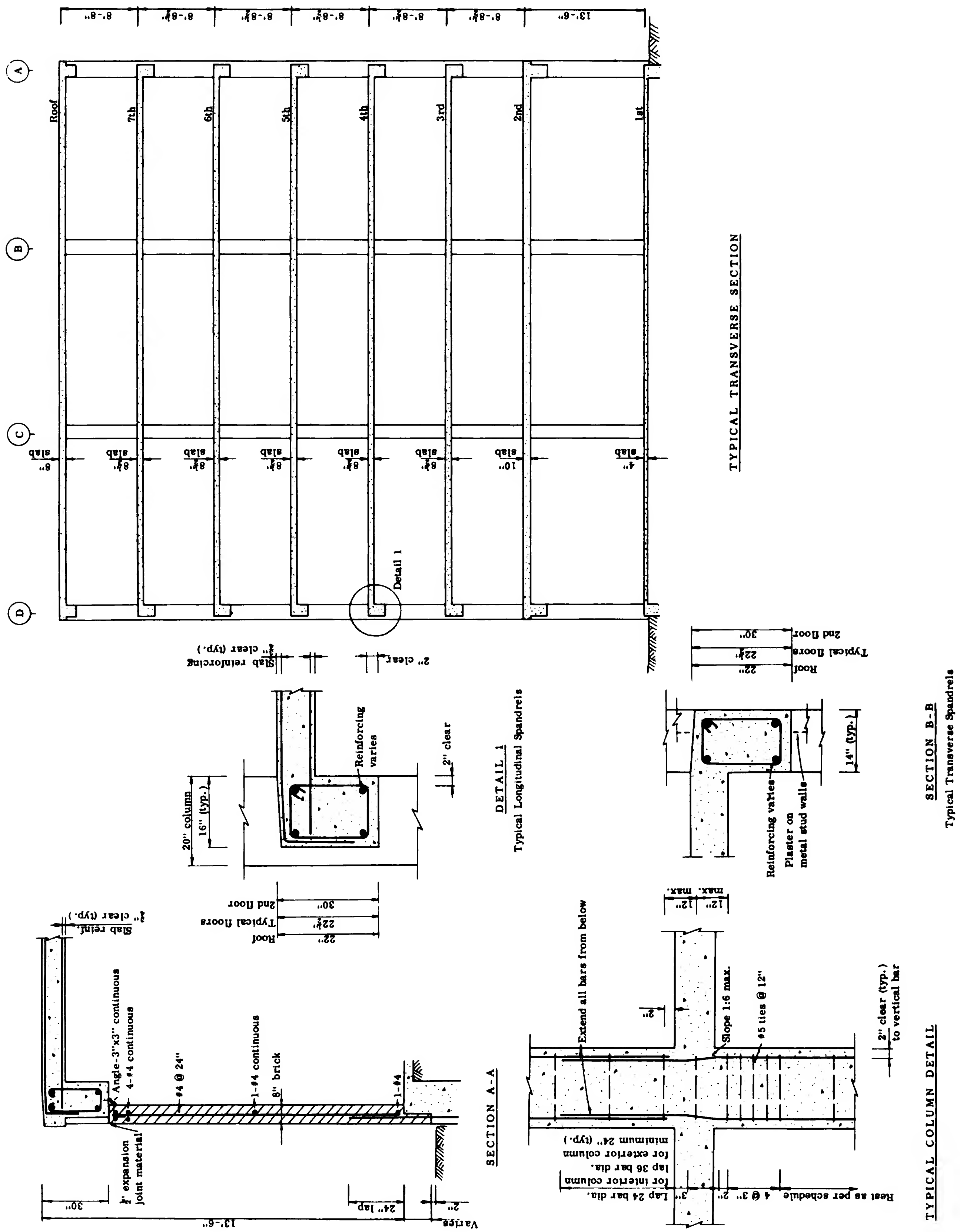

Figure 7.—Holiday Inn, Marengo Street. Sections and details.

Table 2.—Peak recorded accelerations [1]

Station	Transverse (S.52°W.) component	Longitudinal (N.38°W.) component	Vertical component
Roof (8th level)	0.426*g* at 7.91 sec	0.247*g* at 8.47 sec	0.140*g* at 3.47 sec.
4th floor	.261*g* at 7.99 sec	.199*g* at 8.38 sec	.109*g* at 3.47 sec.
1st floor (ground level)	.147*g* at 8.60 sec	.139*g* at 7.84 sec	.086*g* at 8.41 sec.

[1] From digitized listing.

forded the exterior frames by the spandrel beams creates exterior frames that are roughly twice as stiff as interior frames.

With the exception of some light framing members supporting the stairway and elevator openings, the structure is essentially symmetrical. The participation of the nonstructural brick filler walls, and some exterior cement plaster, could cause some asymmetry for lateral motion in the longitudinal direction. However, it has been assumed that the effects of this would be minor.

EARTHQUAKE DAMAGE

It cost approximately $95,000 to repair the damage caused by the San Fernando earthquake. This is roughly 7 percent of the initial construction cost of the building. Structural repair amounted to roughly $2,500 of that figure; the remainder was for nonstructural damage.

The structural repair was required at the intermediate stair landing between the first and second floors at column 1 on column line A. Cracking and spalling occurred at the slab and beam column joints. Exterior plaster had to be removed and reinforcement was exposed (figs. 8 through 12). No other damage was observed.

Nonstructural damage occurred in almost every guest room, although to a lesser degree than in the Holiday Inn on Orion Avenue (Building Report 29). Whereas drywall panels had to be replaced in the Orion structure, the cracks in the Marengo Street structure were smaller and could be repaired. Only nine bathtubs (compared to 45 in the Orion structure) had to be replaced; no water closets (compared to 12 in the Orion structure) had to be replaced. The bathroom damage was less extensive than that at the Orion structure. Windows and doors in every guest room required alignment and adjustment. Some sliding windows tilted in their frames, but no glass was reported broken. Cracks were observed in the exterior plaster.

The damage repair costs averaged approximately $1.50 per square foot of floor space.

RECORDED EARTHQUAKE RESPONSE

Motion caused by the San Fernando earthquake was recorded by Earth Sciences AR–240 strong-motion accelerographs located at the roof, fourth floor, and first floor (ground level). At each location (figs. 4, 5, and 6), motion was recorded along the three principal axes, parallel to the long direction of the building (longitudinal), parallel to the short side of the building (transverse), and vertically. Approximately 40 seconds of motion was recorded for each component of motion at each location. Table 2 gives the peak measured accelerations and their times of occurrence for each recording. These values were obtained from a digitized listing of the records.

Figures 13, 14, and 15 were plotted by computer using the digitized records. From a visual examination of the longitudinal and transverse directions of the fourth-floor and roof-level records, the following observations were made: In the first 5 seconds of motion, the apparent fundamental period in each direction was roughly 0.6 to 0.65 second, but at about 8 seconds the fundamental period appeared to be about 1.1 seconds. This leads to the hypothesis that the elastic limits of some elements in the structure were exceeded between 5 and 8 seconds after the start of motion; thereafter, the structure responded periodically in an inelastic manner. This hypothesis formed the basis for the analysis discussed later in this report.

From the digitized first-floor ground motion, recorded response spectra were determined for various percentages of critical damping. Figures 16, 17, and 18 show the response spectra for the transverse and longitudinal directions at 2 and 10 percent of critical damping and the vertical direction at 2 and 5 percent of critical damping. The response spectra have been plotted on four-way log paper to facilitate the reading of either pseudo-absolute acceleration, pseudo-relative velocity, or relative displacement values.

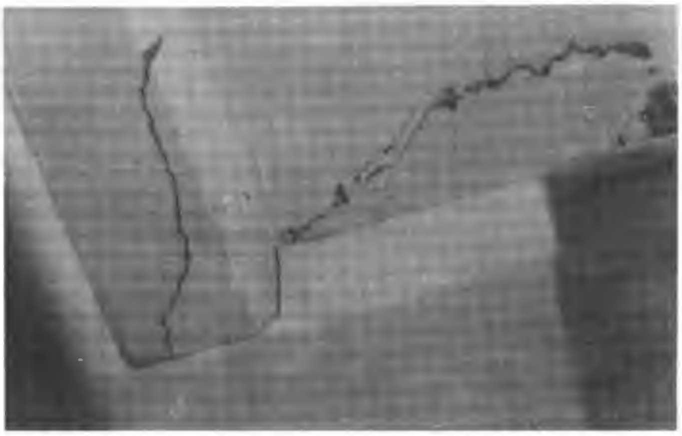

Figure 8.—Holiday Inn, Marengo Street. Interior view of beam and slab soffits at intermediate stair landing. General Adjustment Bureau photograph.

Figure 9.—Holiday Inn, Marengo Street. Same view as figure 8 after damaged concrete has been removed. General Adjustment Bureau photograph.

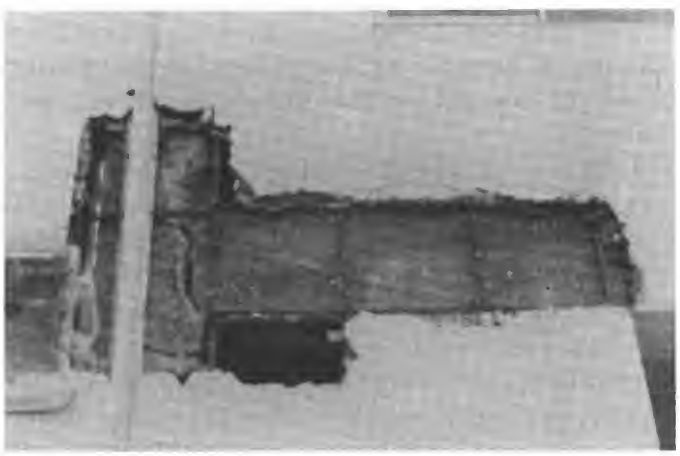

Figure 10.—Holiday Inn, Marengo Street. Exterior view of damage to beam-column joint at the intermediate stair landing. General Adjustment Bureau photograph.

Figure 11.—Holiday Inn, Marengo Street. Closeup view of beam-column joint shown in figure 10 before all the plaster was removed. General Adjustment Bureau photograph.

Figure 12.—Holiday Inn, Marengo Street. Closeup view of figure 11 showing exposed reinforcing bars at beam-column joint. General Adjustment Bureau photograph.

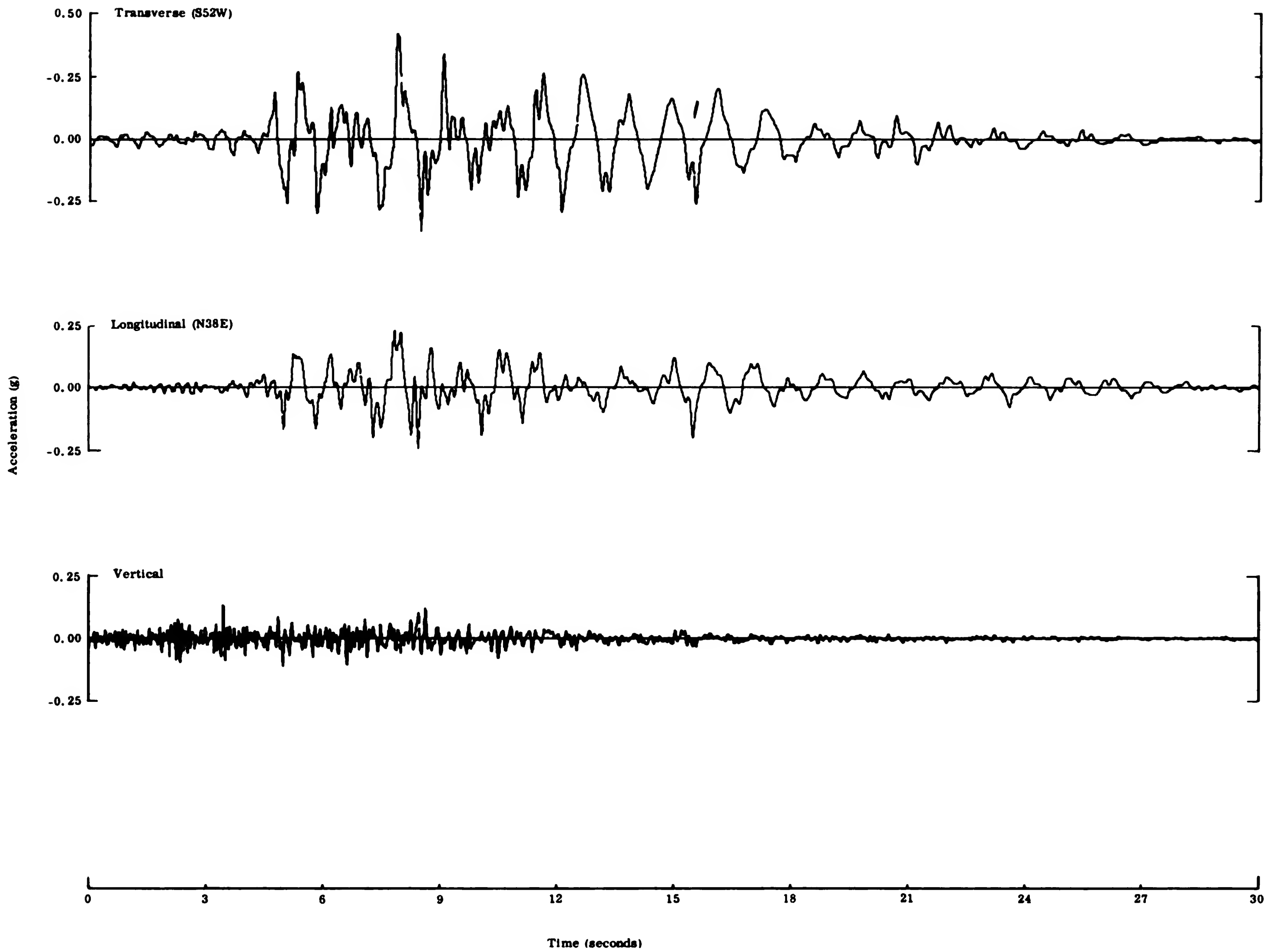

Figure 13.—Holiday Inn, Marengo Street. Recorded acceleration at roof level.

MATHEMATICAL MODELING

The description of analytical procedures outlined the general mathematical modeling procedure. For the Holiday Inn, gross concrete sectional properties (I_o and A_g) were calculated for columns, beams, and slabs. These values were then adjusted, as indicated in table 3, to compensate for the effects of composite beam and slab section, effective slab width, reinforcing steel, the changes of modulus of elasticity, and the differences in effective member lengths. The mathematical models (figs. 19 and 20) include column widths and clear spans of beams. In the vertical direction, the structure was dimensioned for the clear column lengths between slabs and for the slab thickness. Therefore, adjustments only had to be made for the columns at the exterior frames to compensate for the depth of spandrel beams.

Because the study was limited to planar analyses, each mathematical model was two-dimensional, and the building was assumed to be symmetrical with no eccentricity between the center of mass and the center of rigidity. This assumption is considered to be reasonable because, as discussed earlier, this building is essentially symmetrical with the center of mass essentially coincident with the center of rigidity. In addition, the more rigid frames were positioned at all the extremities of the structure, which reduced the effects of accidental torsion. The assumption that the

Table 3.—Member stiffness properties

Member	Moment of inertia	Shear area	Cross-sectional area
Spandrel beams	$1.5\ C_eI_o$		
Slabs	$1.2\ C_eI_o$		
Interior columns	$C_sC_eI_o$	$5/6\ C_eA_g$	C_eA_g
Exterior columns	$C_sC_eC_l{}^3I_o$	$5/6\ C_eC_lA_g$	C_eA_g

$C_e = \sqrt{E/3.3 \times 10^6}$ (See table 1 for values of E).
C_l = Clear-story height between slabs divided by clear-column length between beams. (A value of 1.17 was used for the typical exterior columns.)
C_s = Ratio of moment of inertia of column, including effects of reinforcing steel, to I_o. For models TS1 and LS1, $C_s = 1.0$ because the reinforcement was not considered. For models TS2 and LS2, $C_s = 1.4$, an average of the minimum and maximum calculated values of C_s.

concrete floor slab acts as a rigid horizontal diaphragm is also considered to be reasonable because of the absence of any significant openings, and because the length-to-width ratio is approximately 2.5.

Tables 4 and 5 give descriptions of the various models used in the transverse and longitudinal directions. Models TS1 and LS1 represent the first trial runs. Because the resulting periods appeared long, column reinforcing was added to create models TS2 and LS2. This is justified because the columns are in compression, and therefore the full transformed section can be considered to be effective. Reinforcing was not added to the beams and slabs because these elements were considered cracked sections, which would reduce the effective I_o. It was assumed that the increase due to reinforcement balances the decrease due to cracked sections.

The calculated periods of TS2 and LS2 were still longer than the 0.63- and 0.60-second periods measured during the early portion of the earthquake record. The effects on nonstructural elements were added to the mathematical model by simulating partition elements in the form of diagonal struts. Equivalent elastic stiffness characteristics were obtained from both published (references 11 and 12) and unpublished laboratory test results.

This report does not detail an analytical approach for including the effects of partitions. However, the results of the analysis are presented to give a general approach toward including partitions in the mathematical model. In the transverse direction, the inclusion of partitions in model TS2–P reduced the period slightly, but not enough to match the recorded

Table 4.—Transverse direction mathematical models used in the analysis

Model	Fundamental period	Lateral force-resisting system	Purpose	Earthquake time interval	Number of modes	Applied viscous damping
	Seconds			*Seconds*		*Percent*
TS1	0.93	Bare structural frame	Periods and mode shapes			
TS2	.88	Increase column moments of inertia to include reinforcing steel.	Periods, mode shapes, and member forces from model displacements.			
TS2–P	.84	Add partitions and plaster as equivalent diagonal struts.	Study influence of partitions			
TS2–PW	.54	Model plaster at ends as shear walls.	Study influence of exterior plaster acting as a shear wall.			
TD1	.63	TS2 adjusted for period, $E = 6.4 \times 10^6$ psi.	Dynamic analysis, preyield	0 to 5	3	2
TD2	1.15	TS2 adjusted for period, $E = 1.9 \times 10^6$ psi.	Dynamic analysis, postyield	0 to 24	3	5

Table 5.—Longitudinal direction mathematical models used in the analysis

Model	Fundamental period	Lateral force-resisting system	Purpose	Earthquake time interval	Number of modes	Applied viscous damping
	Seconds			*Seconds*		*Percent*
LS1	0.86	Bare structural frame	Periods and mode shapes			
LS2	.79	Increase column moments of inertia to include reinforcing steel.	Periods, mode shapes, and member forces from modal displacements.			
LS2–PW	.68	Add partitions, plaster, and brick	Study effects of nonstructural elements.			
LS2–PW2	.64	Double stiffness of brick and add adjacent plaster 1st-floor wall.	Study effects of stiffening 1st-floor nonstructural elements.			
LD1	.60	LS2 adjusted for period, $E = 5.7 \times 10^6$ psi.	Dynamic analysis, preyield	0 to 5	3	2
LD2	1.10	LS2 adjusted for period, $E = 1.7 \times 10^6$ psi.	Dynamic analysis, postyield	0 to 24	3	5

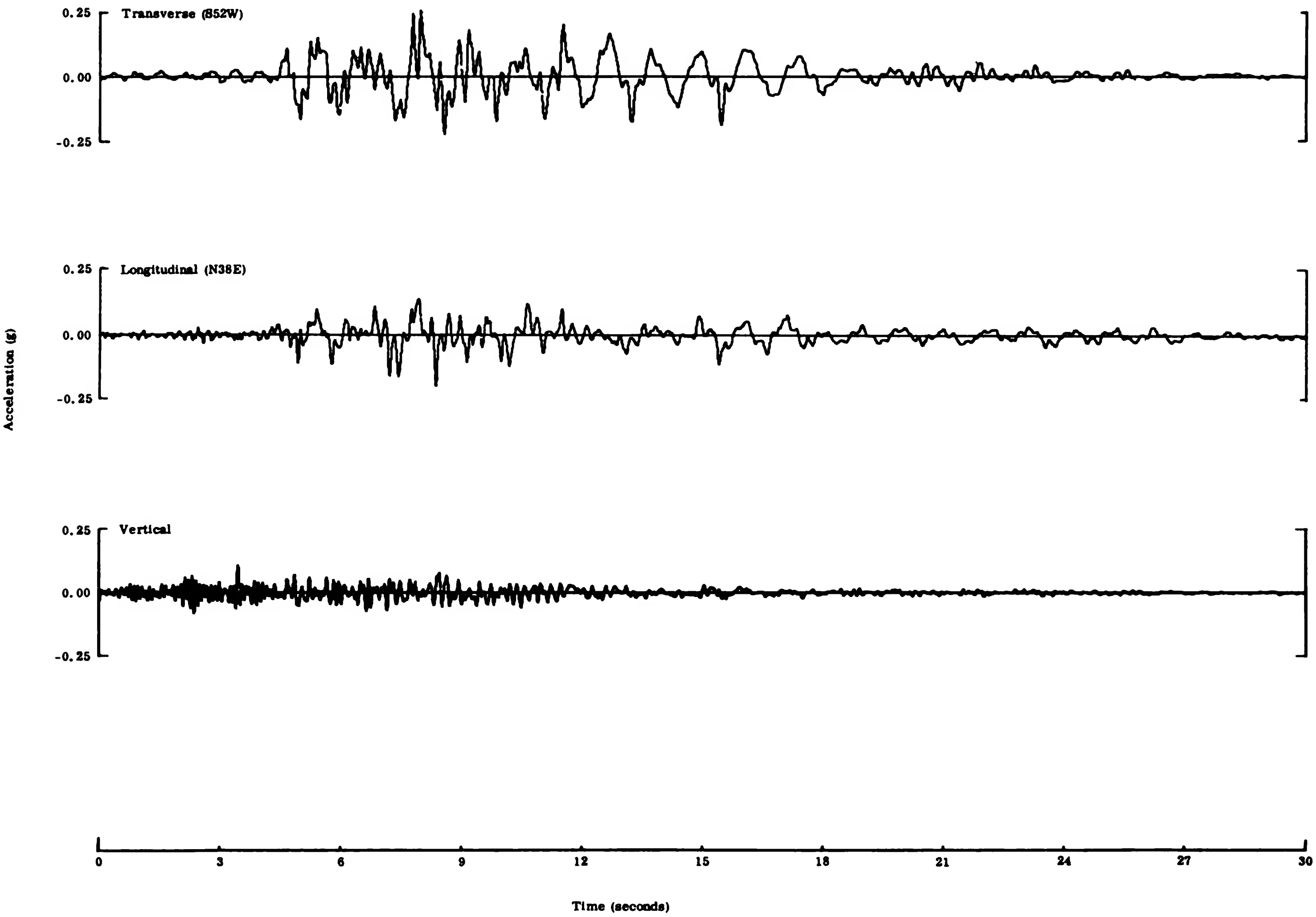

Figure 14.—Holiday Inn, Marengo Street. Recorded acceleration at fourth level.

period. In model TS2–PW, additional stiffness was added by treating the plaster wall at each end as a solid shear wall, ignoring some effects of construction and expansion joints. This significantly affected the reduction of the periods of the structure. Apparently, the response of the structure during the first 5 seconds of the earthquake can be represented by a model somewhere between models TS2–P and TS2–PW.

In the longitudinal model, LS2–PW, the inclusion of interior partitions, exterior plaster at the stair and elevator bay, and the four bays of brick wall at the first floor significantly reduced the period to nearly match the early recorded earthquake motion. Model LS2–PW2 was additionally stiffened by doubling the stiffness characteristics of the brick and by adding the participation of exterior plaster walls at the south canopy area of the first floor. This model appears to more accurately represent the measured period for the earlier portion of the record.

For the dynamic time-history analysis, the bare structural frame representations of models TS2 and LS2 were used as the basic mathematical model. These models were adjusted to match the recorded fundamental periods by changing the value for the modulus of elasticity, E. In effect, this is the same as changing the values of I_o and A_g because the stiffness of the structure is dependent on the products of EI_o and A_gE. By using this method to adjust the natural periods of vibration, the characteristics of the calculated structural model are maintained. This assumes equally distributed softening or hardening

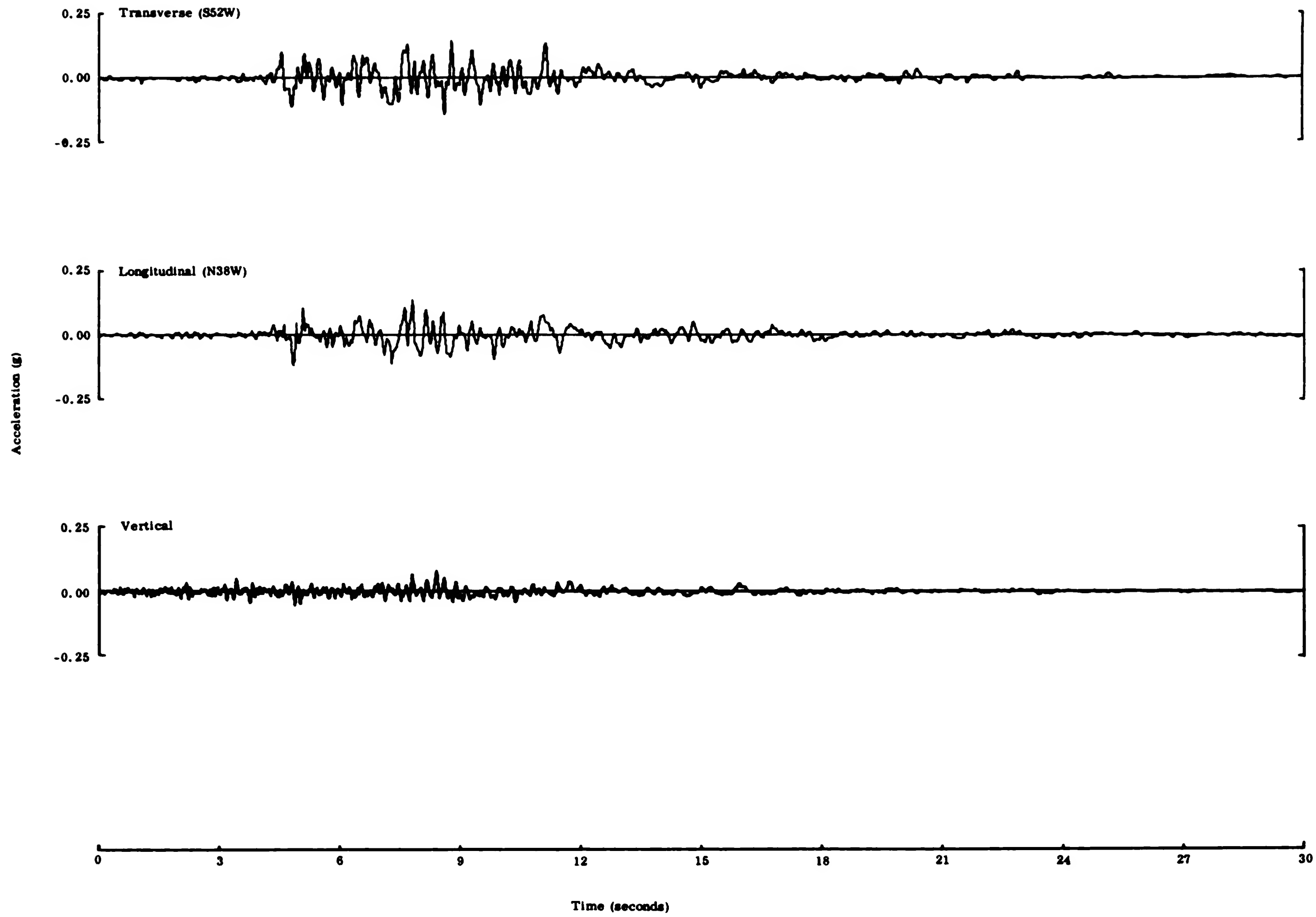

Figure 15.—Holiday Inn, Marengo Street. Recorded acceleration at ground level.

effects between all structural elements. Although this actually may not be correct, it provides an economical alternative to a bilinear or inelastic procedure, which surpasses the scope of this report.

Models TD1 and LD1 represent the structure during the early portions of the earthquake. Models TD2 and LD2 represent the structure during the latter part of the record. The periods and damping values were obtained by analyses, correlating recorded and computed acceleration response.

RESULTS OF ANALYSIS

The previous section described the mathematical models developed for each principal direction. Each model was developed from a careful assessment of the anticipated actual stiffness characteristics and mass distribution of the structure. Review of the recorded motion records (figs. 13 and 14) indicated that the building, during approximately the first 5 seconds of the earthquake, responded at shorter periods than that indicated by the fundamental periods of the basic mathematical models, TS2 and LS2. These differences appear to be due to the participation of nonstructural elements, such as drywall partitions, exterior plaster walls, and infill brick walls. For the latter part of the earthquake, the structure responded at longer periods than those indicated by the basic mathematical models. Apparently, these differences were due to a loss of stiffness caused by some structural members exceeding the elastic capacity.

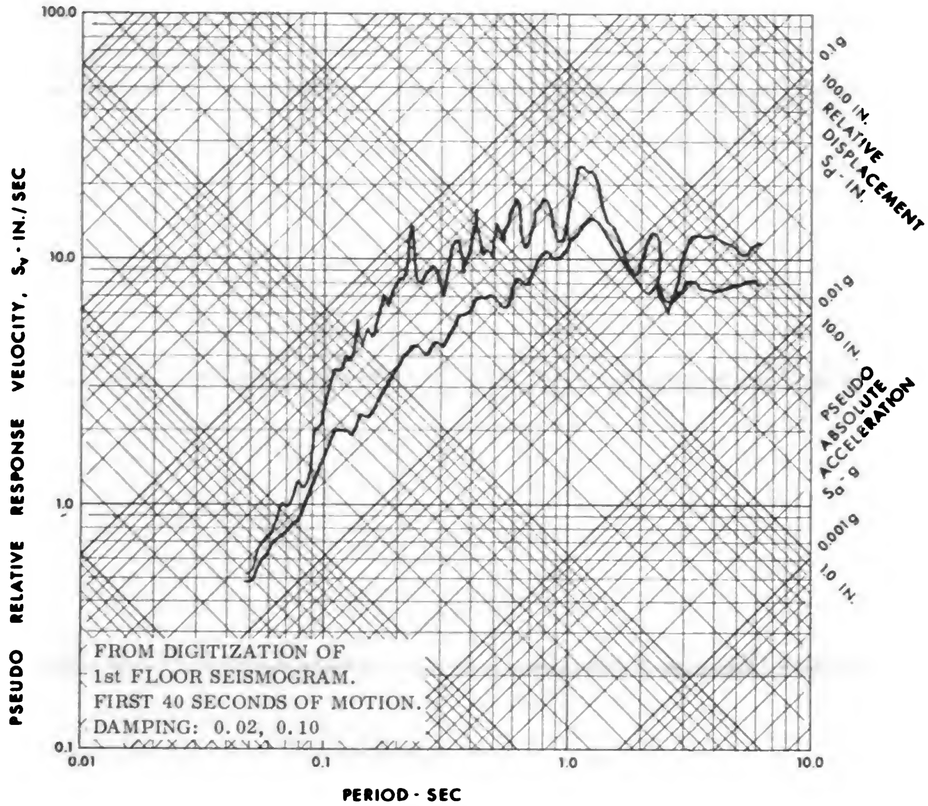

Figure 16.—Holiday Inn, Marengo Street. Transverse response spectra.

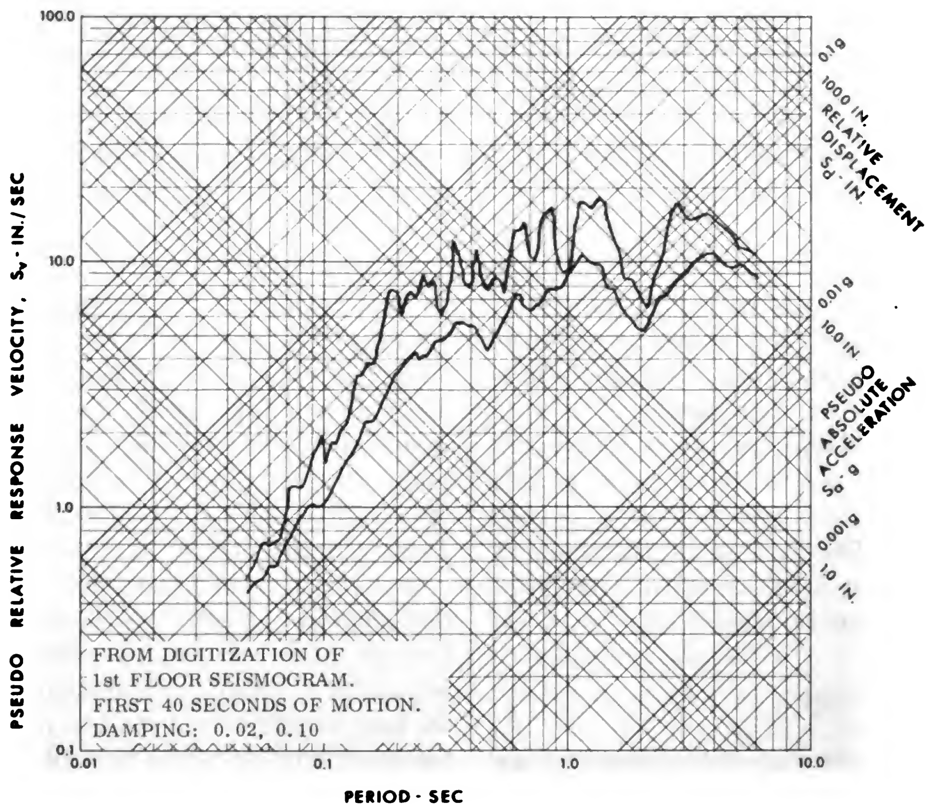

Figure 17.—Holiday Inn, Marengo Street. Longitudinal response spectra.

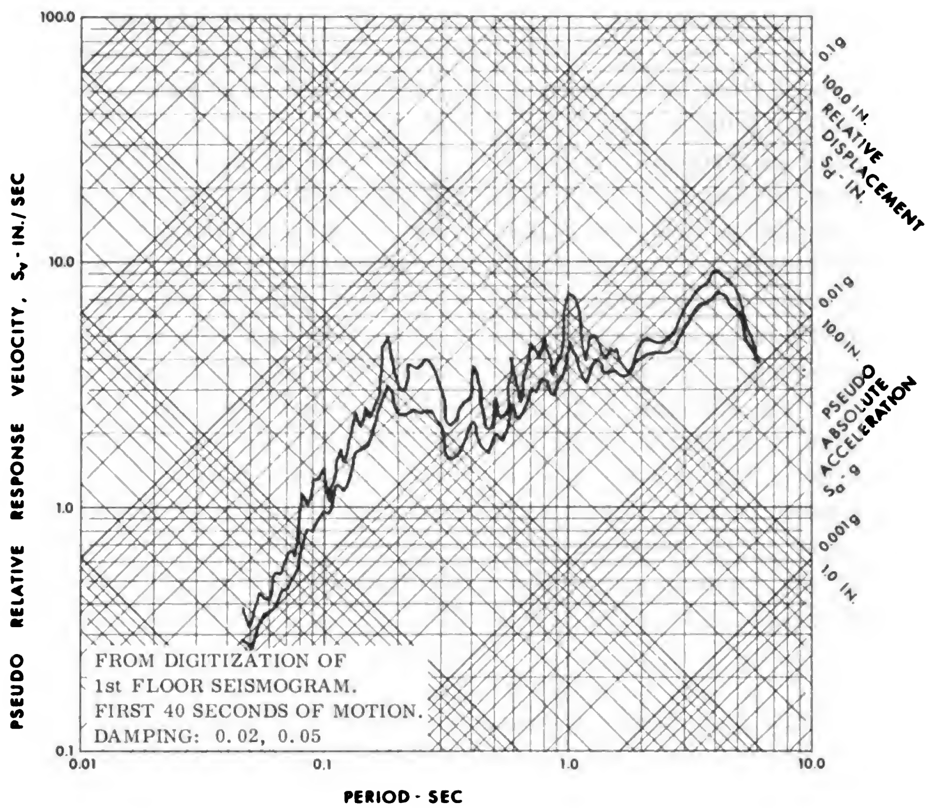

Figure 18.—Holiday Inn, Marengo Street. Vertical response spectra.

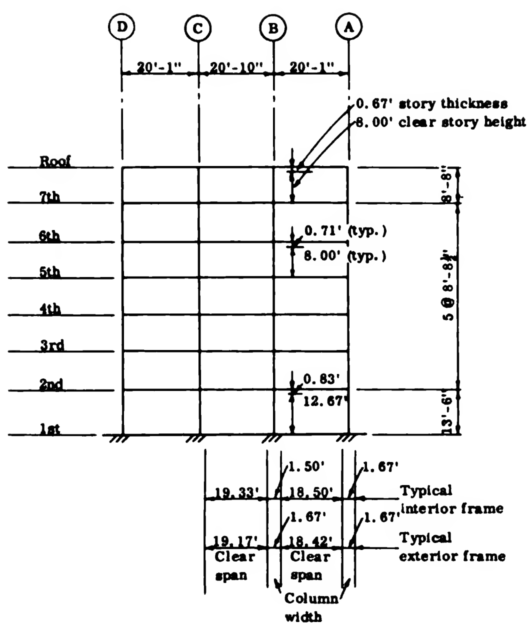

Figure 19.—Holiday Inn, Marengo Street. Typical transverse frame.

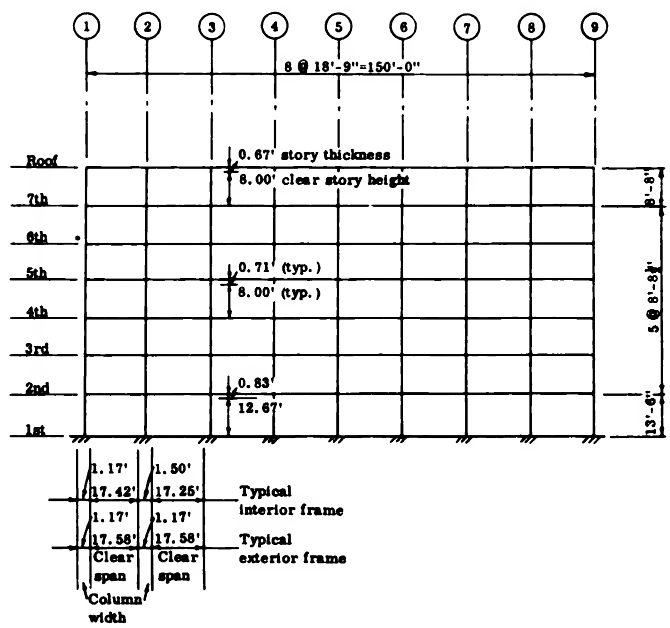

Figure 20.—Holiday Inn, Marengo Street. Typical longitudinal frame.

Mode Shapes and Periods of Vibration

Mode shapes and periods of vibration were calculated for the first seven translational modes in both the transverse and longitudinal directions (tables 6 and 7). A review of these modes, in conjunction with the response spectra, shows that the higher modes are progressively less responsive to the earthquake. Therefore, the analysis only considered the first three translational modes in each direction. Figure 21 shows the masses, periods, and mode shapes for the three modes of models TS2 and LS2. The mode shapes are normalized at the roof to give the relative shapes between modes. Tables 6 and 7 show numerical values for the same models. These mode shapes and masses also apply to the transverse models, TD1 and TD2, and the longitudinal models, LD1 and LD2. Therefore, the periods for the higher modes of these models can be obtained by proportioning the higher mode periods to the fundamental periods in the same ratio as the TS2 and LS2 models.

The values given in tables 6 and 7 for the ratios of effective V/W to spectral acceleration are useful for spectral analysis. When multiplied by the appropriate spectral acceleration, these ratios (effective modal loads) will give the equivalent base shear coefficient. In the case of the first mode, this value can be compared with the UBC design coefficient, ZKC.

For example, take the value of 0.86 from table 7. Multiply it by the spectral acceleration of 0.15*g* for a period of 0.79 second and a damping ratio of 10 percent (fig. 17). This gives a value of 0.13 for ZKC, which can be compared to the design code requirements of 0.04. This is discussed later (see also table 10).

Table 6.—Mode shapes and periods, transverse direction, model TS2

Mode number		1	2	3	4	5	6	7
Period of vibration (seconds)		0.880	0.288	0.164	0.106	0.073	0.055	0.046
Story	Mass	Mode shapes						
	kips-sec²/ft							
Roof	43.7800	0.0794	0.0747	0.0684	−0.0588	−0.0439	−0.0273	−0.0123
7th	45.3400	.0745	.0411	−.0040	.0501	.0768	.0703	.0382
6th	45.3400	.0666	−.0042	−.0644	.0635	−.0085	−.0740	−.0623
5th	45.3400	.0558	−.0471	−.0630	−.0309	−.0735	.0260	.0757
4th	45.3400	.0425	−.0718	−.0023	−.0740	.0469	.0399	−.0763
3d	45.3400	.0279	−.0697	.0604	.0052	.0500	−.0785	.0639
2d	56.8300	.0149	−.0467	.0677	.0676	−.0575	.0447	.0272
	Σ = 327.3100							
Effective V/W ÷ spectral acceleration		.83	.12	.04	.01	.00	.00	.00
Modal roof acceleration ÷ spectral acceleration		1.31	−.47	.24	−.11	.05	−.02	.00

Table 7.—Mode shapes and periods, longitudinal direction, model LS2

Mode number		1	2	3	4	5	6	7
Period of vibration (seconds)		0.791	0.266	0.156	0.104	0.076	0.060	0.052
Story	Mass	Mode shapes						
	kips-sec²/ft							
Roof	43.7800	0.0765	0.0719	0.0684	−0.0614	−0.0476	0.0305	0.0139
7th	45.3400	.0728	.0450	.0045	.0439	.0762	−.0731	−.0403
6th	45.3400	.0663	.0037	−.0599	.0702	−.0002	.0718	.0630
5th	45.3400	.0568	−.0388	−.0698	−.0211	−.0764	−.0222	−.0754
4th	45.3400	.0450	−.0673	−.0165	−.0774	.0417	−.0423	.0756
3d	45.3400	.0315	−.0722	.0515	−.0054	.0544	.0786	−.0634
2d	56.8300	.0191	−.0549	.0700	.0638	−.0530	−.0419	.0261
	Σ = 327.3100							
Effective V/W ÷ spectral acceleration		.86	.10	.03	.01	.00	.00	.00
Modal roof acceleration ÷ spectral acceleration		1.28	−.42	.20	−.09	.04	−.01	.00

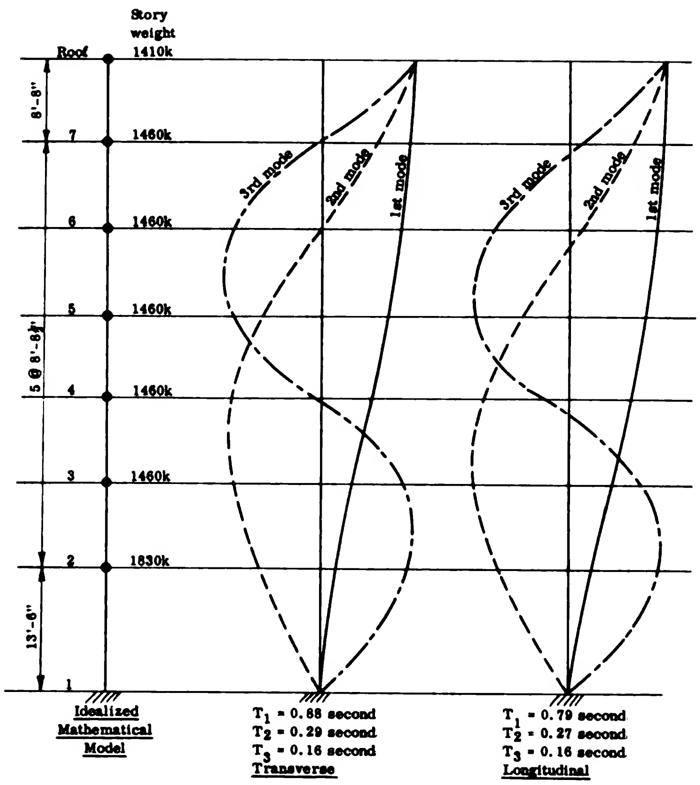

Figure 21.—Holiday Inn, Marengo Street. Calculated periods and mode shapes.

Naturally, using different fundamental periods will yield different values. In a manner similar to that just described, spectral analysis can be aided by using the ratios for modal roof acceleration to spectral acceleration (tables 6 and 7). In these situations, one can calculate the peak roof acceleration for any mode.

Computed Floor Accelerations

The dynamic analyses procedure included calculation and plotting of accelerations for the roof and fourth-floor levels of the structure. These calculations were made by using the recorded first-floor motion, the SMIS computer program, the measured fundamental periods, and the calculated mode shapes and modal period relationships from models TS2 and LS2. The paper on analytical procedures in this volume describes these procedures.

Figures 22 and 23 show both calculated and recorded acceleration time histories. In each group of three plots, the center plot is the recorded acceleration, the top plot is the calculated acceleration for comparison with the early portion of the record (first 5 seconds), and the bottom plot is the calculated acceleration for comparison with the latter part of the earthquake record (after 8 seconds).

Tables 8 and 9 compare recorded and computed peak roof and fourth-floor accelerations. The FRMDYN model data show the peak maximum values for the combined first three modes, the time of occurrence, and the contribution of each mode. The response spectra analysis shows the peak value for each mode of the structure, assuming it had remained elastic during 40 seconds of motion. These peaks do not necessarily occur simultaneously. The sum of the three modes indicates what the maximum value could theoretically be if all three modes peaked at the same time. The root-mean-square (RMS) indicates a statistical mean value for the combination of the three modes. The peak measurement values and their times of occurrence were obtained from the digitized listing.

For the shorter period models, the response spectra analysis is invalid for comparison with the FRMDYN data because the peak responses of the response spectra occur after the 5-second duration of the FRMDYN model had ended. However, the response spectra summations and root-mean-square accelerations can be compared with the peak accelerations of the calculated time-history roof and fourth-floor plots shown in figures 22 and 23.

In tables 8 and 9, the FRMDYN model results reasonably agree with the measured results. Differences can be attributed to the phase relationship between different modes. This affects the summation of modal contributions.

For the longer period models, the response spectra results can be compared with the FRMDYN and measured results. The FRMDYN results for the combined three modes and the peak measured results will be equal to or greater than the first-mode response spectra results, less than the summation of the response spectra results, and greater than or less than the root-mean-square response spectra results.

Maximum Building Displacements

Figures 24a and 25a show envelopes of maximum total displacement. These represent the peak displacement of each story with respect to the first floor. Not all of the maximums occurred simultaneously. For the shorter period models (TD1 and LD1), maximums occurred approximately 5 seconds after the start of motion. Maximums occurred for the longer period models (TD2 and LD2) beyond 8 sec-

Table 8.—Maximum transverse accelerations and displacements at the roof and the fourth floor

	Mode(s)	Period	Damping	Accelerations		Displacements	
				Roof	4th	Roof	4th
		Seconds	*Percent critical*	*g*	*g*	*Feet*	*Feet*
FRMDYN model TD1 calculated for first 5 seconds of motion.	1st	0.63	2	0.271	0.182	0.086	0.047
	2d	.206	2	.010	−.032	.000	.000
	3d	.117	2	.035	−.025	.000	.000
	3 modes			.282	.180	.086	.047
	[time] [1]			[4.92]	[4.86]	[4.92]	[4.92]
Response spectrum analysis, FRMSTC model TS2 with TD1 periods, first 40 seconds of motion.	1st	.63	2	.59	.32	.19	.10
	2d	.206	2	.30	.29	.01	.01
	3d	.117	2	.12	.00	.00	.00
	SUM			1.01	.61	.20	.11
	RMS [2]			.67	.43	.19	.10
Peak measurement during first 5 seconds	All			.265	.17		
	[time] [1]			[5.07]	[4.99]		
FRMDYN model TD2 calculated for first 24 seconds of motion.	1st	1.15	5	0.30	0.16	0.35	0.19
	2d	0.377	5	.06	.06	.00	.00
	3d	.214	5	.04	.00	.00	.00
	3 modes			.39	.26	.35	.19
	[time] [1]			[7.92]	[8.64]	[8.04]	[8.04]
Response spectrum analysis, FRMSTC model TS2 with TD2 periods, first 40 seconds of motion.	1st	1.15	5	.33	.18	.36	.19
	2d	.377	5	.17	.16	.02	.02
	3d	.214	5	.11	.03	.00	.00
	SUM			.60	.37	.38	.21
	RMS [2]			.38	.24	.36	.19
Peak measurement in 40 seconds	All			.426	.261		
	[time] [1]			[7.91]	[7.99]		

[1] Brackets indicate time from start of motion (expressed in seconds). [2] Root-mean-square.

Table 9.—Maximum longitudinal accelerations and displacements at the roof and the fourth floor

	Mode(s)	Period	Damping	Accelerations		Displacements	
				Roof	4th	Roof	4th
		Seconds	*Percent critical*	*g*	*g*	*Feet*	*Feet*
FRMDYN model LD1 calculated for first 5 seconds of motion.	1st	0.60	2	0.034	0.05	0.023	0.013
	2d	.201	2	.18	.07	.003	.003
	3d	.118	2	.16	.00	.00	.00
	3 modes			.06	.135	.020	.162
	[time] [1]			[4.86]	[4.92]	[4.92]	[4.92]
Response spectrum analysis, FRMSTC model LS2 with LD1 periods, first 40 seconds of motion.	1st	.60	2	.38	.23	.11	.07
	2d	.201	2	.25	.24	.008	.008
	3d	.118	2	.06	.01	.00	.00
	SUM			.69	.48	.12	.08
	RMS [2]			.46	.33	.11	.07
Peak measurement during first 5 seconds	All			.17	.11		
	[time] [1]			[5.02]	[4.96]		
FRMDYN model LD2 calculated for first 24 seconds of motion.	1st	1.10	5	.24	.12	.23	.14
	2d	.371	5	.08	.06	.00	.00
	3d	.217	5	.015	.05	.00	.00
	3 modes			.26	.18	.23	.14
	[time] [1]			[7.92]	[8.52]	[7.92]	[7.92]
Response spectrum analysis, FRMSTC model LS2 with LD2 periods, first 40 seconds of motion.	1st	1.10	5	.26	.15	.26	.15
	2d	.371	5	.15	.14	.02	.016
	3d	.217	5	.08	.02	.003	.001
	SUM			.49	.31	.28	.17
	RMS [2]			.31	.21	.26	.15
Peak measurement in 40 seconds	All			.25	.20		
	[time] [1]			[8.47]	[8.38]		

[1] Brackets indicate time from start of motion (expressed in seconds). [2] Root-mean-square.

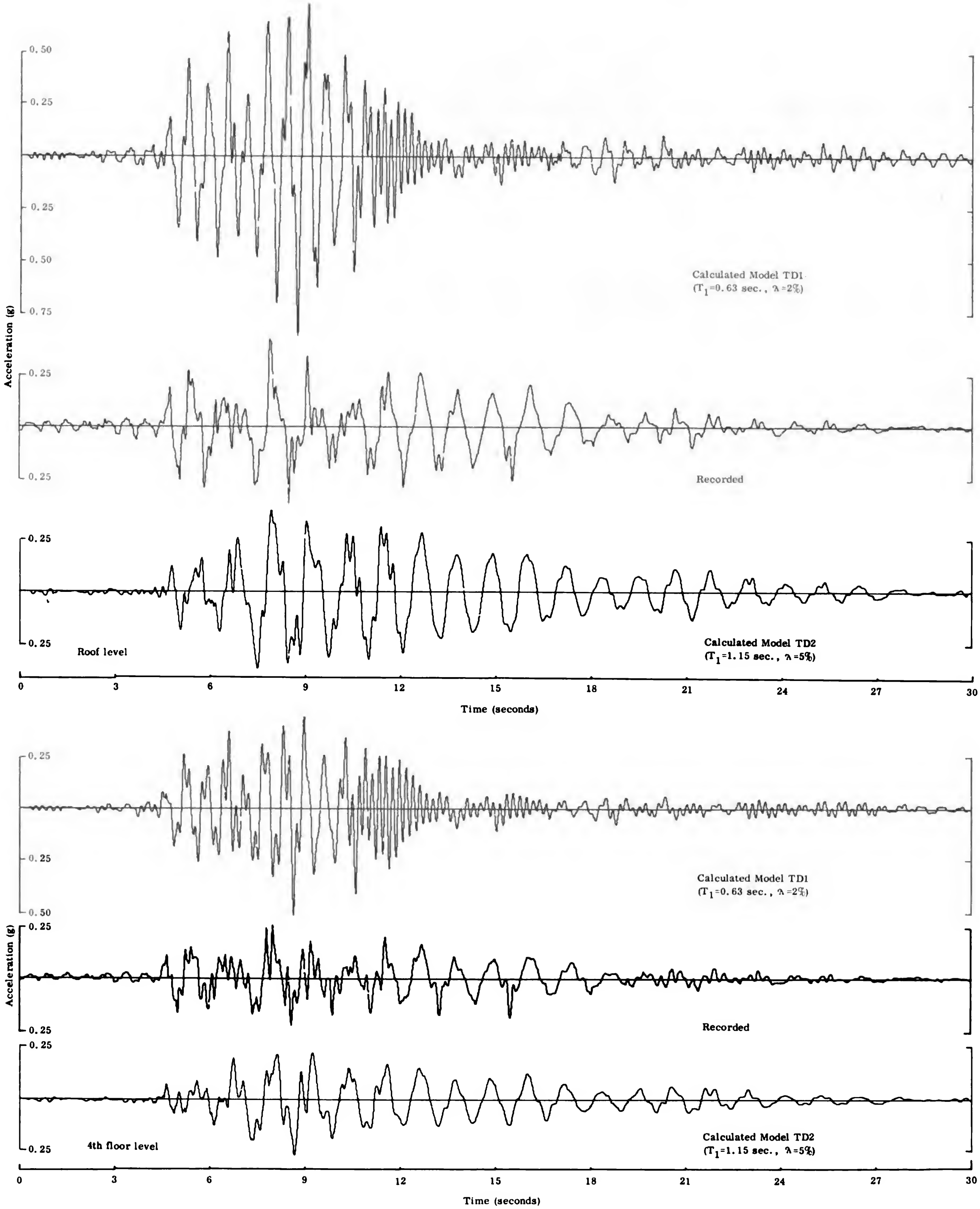

Figure 22.—Holiday Inn, Marengo Street. Transverse calculated and recorded accelerations.

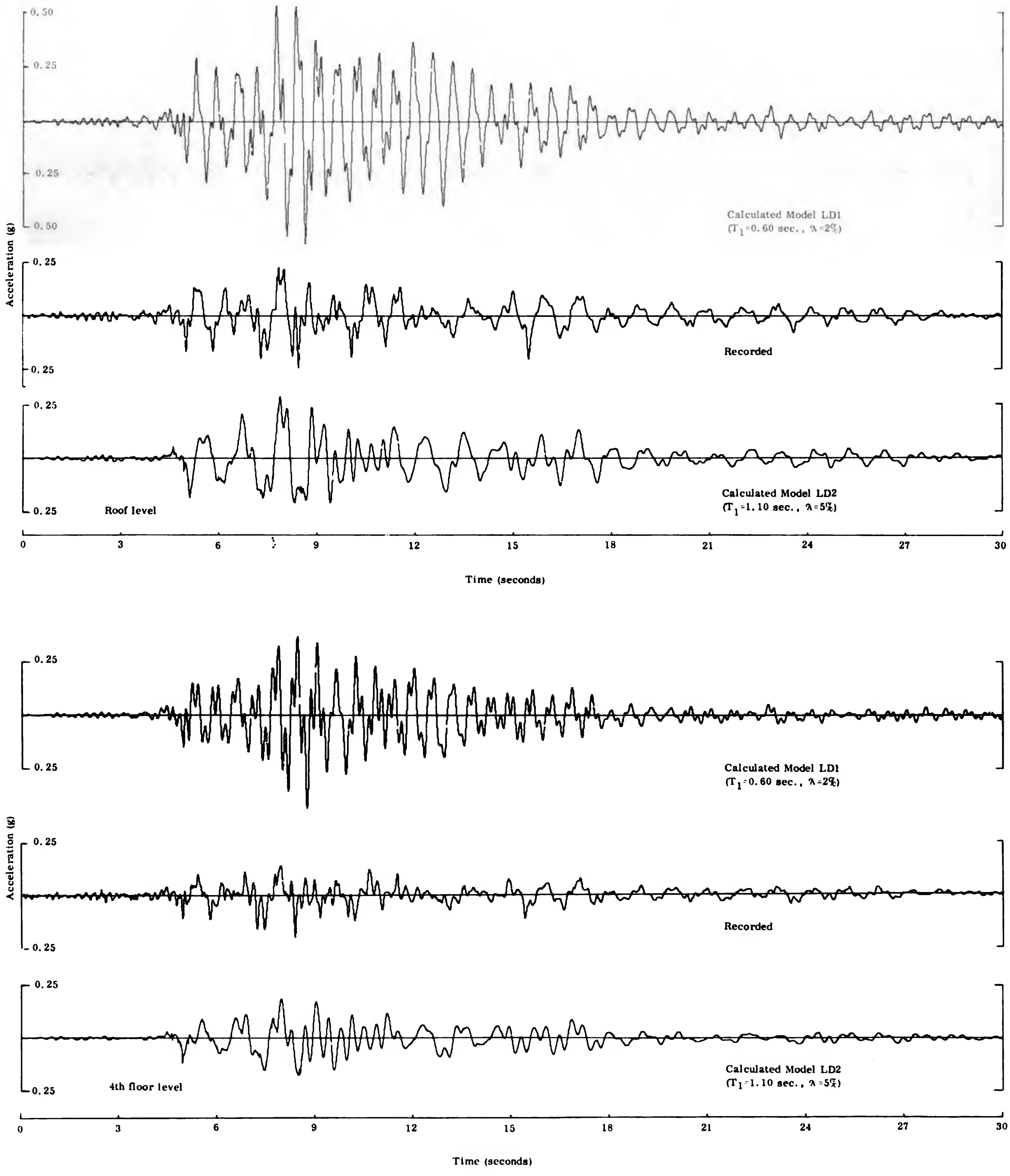

Figure 23.—Holiday Inn, Marengo Street. Longitudinal calculated and recorded accelerations.

onds after the start of motion. Times for maximum roof and fourth-floor displacements are shown in tables 8 and 9. Figures 24a and 25a also show maximum interstory drifts.

Maximum Story Forces

Figures 24b and 25b plot the maximum horizontal story forces from the dynamic analysis and corresponding code values. For each level, story forces were determined as the product of the story mass times the maximum absolute story acceleration.

Maximum Story Shears

Maximum story shears were calculated as part of the dynamic analysis. Figures 24c and 25c indicate these as smooth curves with corresponding code values. Code design shears have been computed for the requirements of both the 1967 UBC, which was equivalent to the one used in the original design, and for the 1970 UBC, which would govern if the structure were to be built in 1971. The UBC story shears are the same for both directions. They are based on the numerical coefficients and periods shown in table 10.

Table 10.—UBC seismic design parameters

Code	Z	K	C	Base shear V = ZKCW	Period (T)	J
					Second	
1967 UBC...........	1.00	0.67	0.057	0.038 W	0.70	0.64
1970 UBC...........	1.00	1.00	.057	.057 W	.70	.77

The dynamic analysis determined peak values for the shears. These were calculated to have occurred at each floor level of the model at some time during the time-history response. These peak values did not necessarily occur simultaneously. They should be considered to represent only an envelope of maximum story shears.

The total response story shears were determined from the contribution of the first three modes of vibration, with the first-mode contribution accounting for much of the total response. Figures 24c and 25c also show the contributions of the first mode at the time of maximum story shears. The second- and third-mode contributions were too small to show in the figures.

Maximum Overturning Moments

Envelopes of maximum overturning moments were calculated in a manner similar to that used to determine the maximum story shears. Figures 24d and 25d show these with corresponding 1967 and 1970 UBC values (except $J = 1.0$ was used for the 1970 UBC values). The first three modes of vibration were determined in the same way as the maximum story shears. Figures 24d and 25d also show the contributions of the significant modes to total response at the time of maximum response.

After publication of the 1970 UBC, the J factor for determining the base overturning moment received considerable scrutinization. Table 10 indicates that the 1970 UBC would require a minimum J value of 0.77, but subsequent amendments to the 1970 UBC have increased the minimum J value to 1.0. For this reason, overturning moments determined with a value of $J = 1.0$ have been used in figures 24d and 25d as a comparison with the 1967 UBC minimums in effect at the time the building was designed.

Loads on Key Structural Elements

Earthquake loads on structural elements in the lateral force-resisting frames were investigated by comparing seismic and estimated vertical load effects with the estimated capacity for several representative members. Hand analysis approximated the vertical dead and live load forces. The FRMDYN computer program produced the seismic forces.

As stated earlier, the models used in the FRMDYN computer runs were adjusted to respond at the measured periods. This was done by changing the modulus of elasticity, E, to change the stiffness.

In the shorter period models the value of E was increased. Apparently, part of this increase was used to offset the effects of the partitions. Also, part of the increase accounted for the nonlinearity of reinforced concrete, which has a higher E at lower amplitudes of structural strain.

The value of E was decreased in the longer period models. This change of E can be accounted for by a combination of nonlinearity at higher strains, loss of stiffness due to cracked sections, and inelastic behavior of some yielded elements. The values shown in tables 11 through 14 must be interpreted with the above remarks kept in mind.

Table 11.—Summary of girder $\frac{M}{M_u}$ for transverse frames

Frame	Floor	Reinforcement location	Model TD1 Exterior joint	Model TD1 1st interior joint	Model TD1 2d interior joint	Model TD2 Exterior joint	Model TD2 1st interior joint	Model TD2 2d interior joint
Transverse exterior frame	Roof	Top bars	1.7	1.0	1.3	2.1	1.2	1.6
		Bottom bars	0.5	0.1	0.7	0.8	0.4	1.1
	4th	Top bars	2.0	1.8	1.8	2.5	2.2	2.2
		Bottom bars	2.5	2.3	2.9	3.4	3.0	3.7
	2d	Top bars	2.5	2.3	2.1	2.9	2.7	2.4
		Bottom bars	3.4	2.8	3.0	4.1	3.4	3.6
Transverse interior frame	Roof	Top bars	.9	.5	.5	1.1	.5	.5
		Bottom bars				.1		
	4th	Top bars	1.2	1.2	1.1	1.5	1.4	1.4
		Bottom bars	1.4	1.0	1.1	1.8	1.5	1.5
	2d	Top bars	1.4	1.1	1.1	1.7	1.3	1.2
		Bottom bars	1.9	1.5	1.5	2.4	2.0	2.0

Table 12.—Summary of girder $\frac{M}{M_u}$ for longitudinal frames

Frame	Floor	Reinforcement location	Model LD1 Exterior joint	Model LD1 1st interior joint	Model LD1 2d interior joint	Model LD2 Exterior joint	Model LD2 1st interior joint	Model LD2 2d interior joint
Longitudinal exterior frame	Roof	Top bars	0.6	0.5	0.5	1.0	0.6	0.8
		Bottom bars				0.1		
	4th	Top bars	.7	.7	.7	1.7	1.7	1.7
		Bottom bars	.3	.1	.1	2.0	1.5	2.6
	2d	Top bars	1.2	1.0	.8	2.5	1.8	1.6
		Bottom bars	1.0	.5	.6	2.6	1.7	2.5
Longitudinal interior frame	Roof	Top bars	.6	.4	.4	.8	.5	.5
		Bottom bars						
	4th	Top bars	.8	.5	.5	1.6	.8	.9
		Bottom bars				.8	.5	.6
	2d	Top bars	.9	.6	.6	1.7	2.0	1.1
		Bottom bars	.3	.1	.1	1.2	1.0	1.5

Table 13.—Summary of column interaction for transverse frames

Column and floor level	Model TD1 $\frac{M_x}{M_{ux}}+\frac{M_y}{M_{uy}}$	Model TD1 $\frac{P}{P_{uo}}$	Model TD2 $\frac{M_x}{M_{ux}}+\frac{M_y}{M_{uy}}$	Model TD2 $\frac{P}{P_{uo}}$
Typical exterior column:				
Roof	0.5	0.03	0.5	0.03
4th	.5	.1	.6	.1
2d	.7	.1	.8	.1
Typical interior column:				
Roof	.4	.06	.5	.06
4th	.5	.2	.7	.2
2d	.8	.2	.9	.2
Typical corner column:				
Roof	.8	.01	1.0	.01
4th	1.8		2.3	[1] .1
2d	2.2	[1] .1	2.8	[1] .2

[1] Column axial load in tension.

Table 14.—Summary of column interaction for longitudinal frames

Column and floor level	Model LD1 $\frac{M_x}{M_{ux}}+\frac{M_y}{M_{uy}}$	Model LD1 $\frac{P}{P_{uo}}$	Model LD2 $\frac{M_x}{M_{ux}}+\frac{M_y}{M_{uy}}$	Model LD2 $\frac{P}{P_{uo}}$
Typical exterior column:				
Roof	0.3	0.03	0.6	0.03
4th	.5	.2	1.3	.2
2d	.5	.2	1.1	.1
Typical interior column:				
Roof	.04	.05	.2	.06
4th	.2	.2	.4	.2
2d	.4	.2	.8	.2
Typical corner column:				
Roof	.5	.02	.7	.02
4th	.4	.1	1.3	.05
2d	.7	.08	1.5	.02

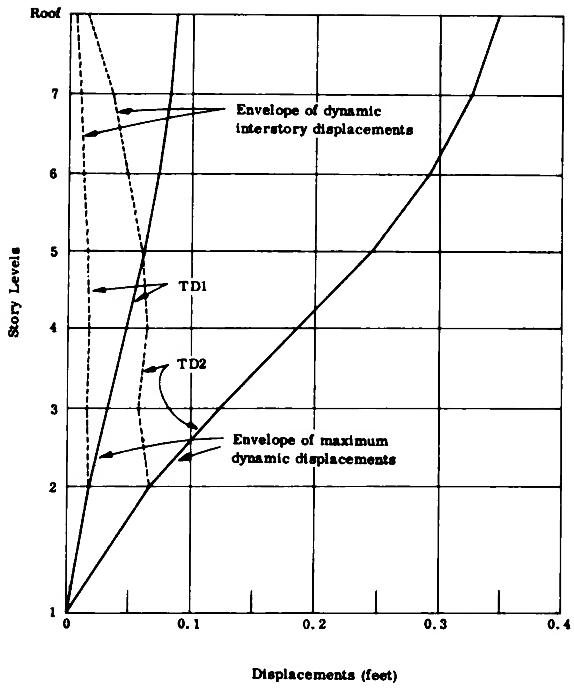

Figure 24a. TOTAL BUILDING DISPLACEMENTS AND INTERSTORY DISPLACEMENTS

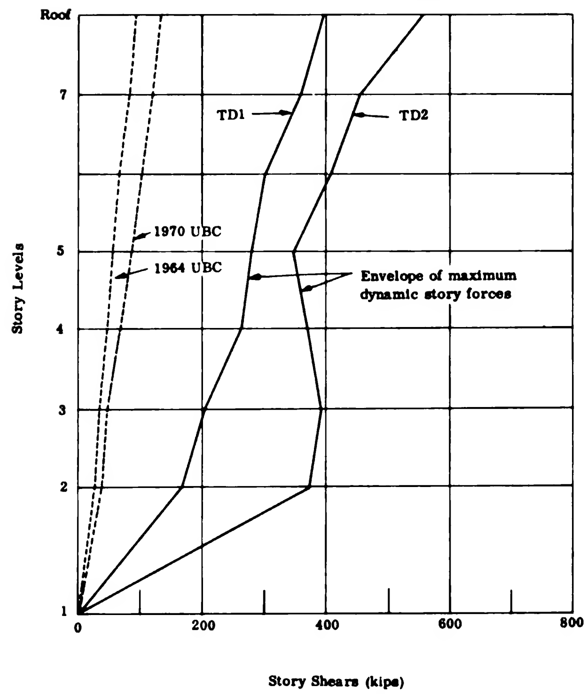

Figure 24b. MAXIMUM STORY FORCES

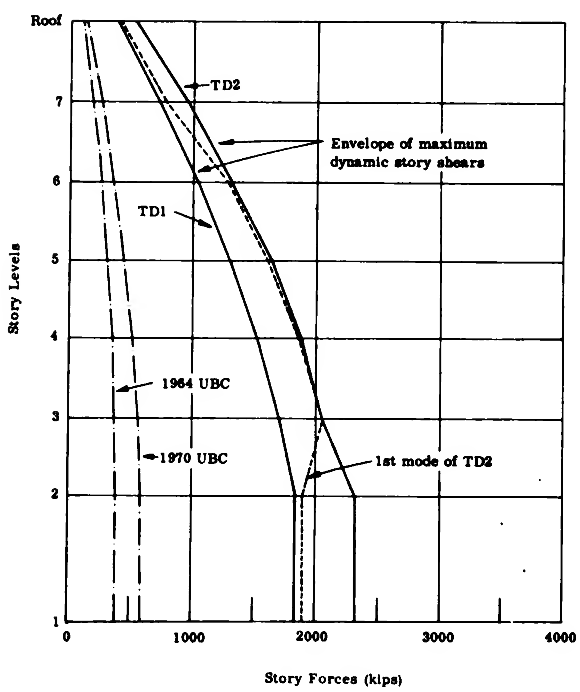

Figure 24c. MAXIMUM STORY SHEARS

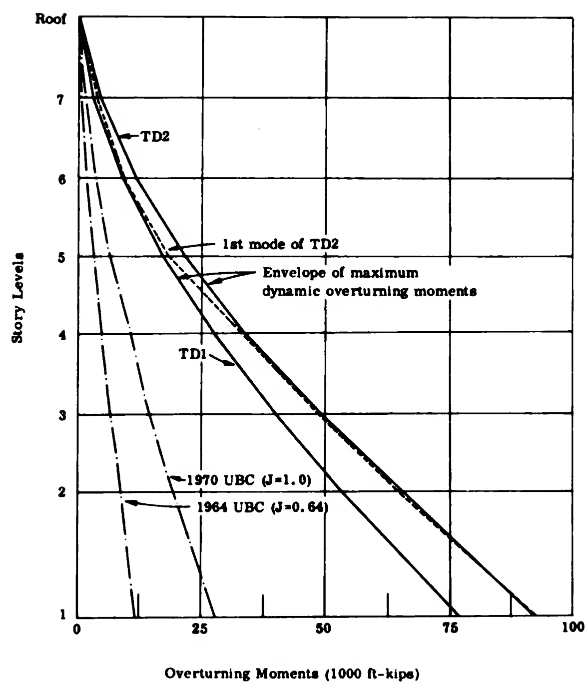

Figure 24d. MAXIMUM OVERTURNING MOMENTS

Figure 24.—Holiday Inn, Marengo Street. Dynamic response and design code values for transverse (52° W.) direction.

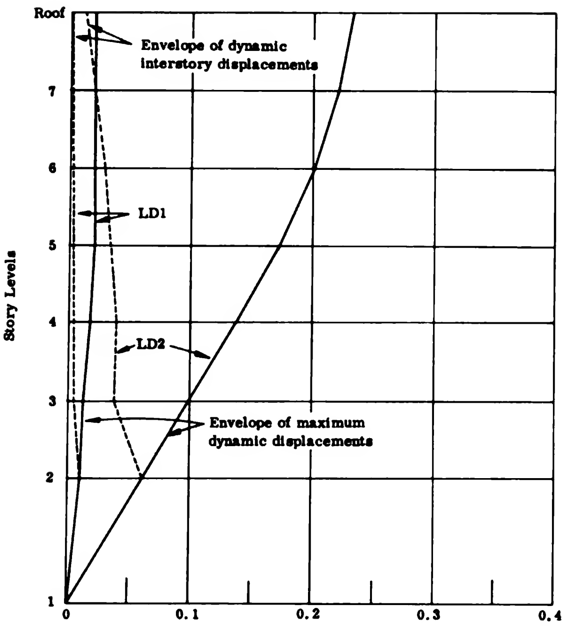

Figure 25a. TOTAL BUILDING DISPLACEMENTS AND INTERSTORY DISPLACEMENTS

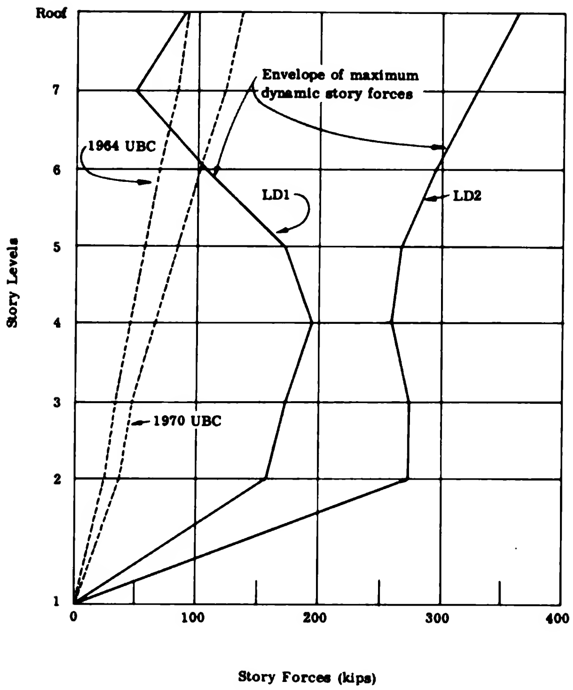

Figure 25b. MAXIMUM STORY FORCES

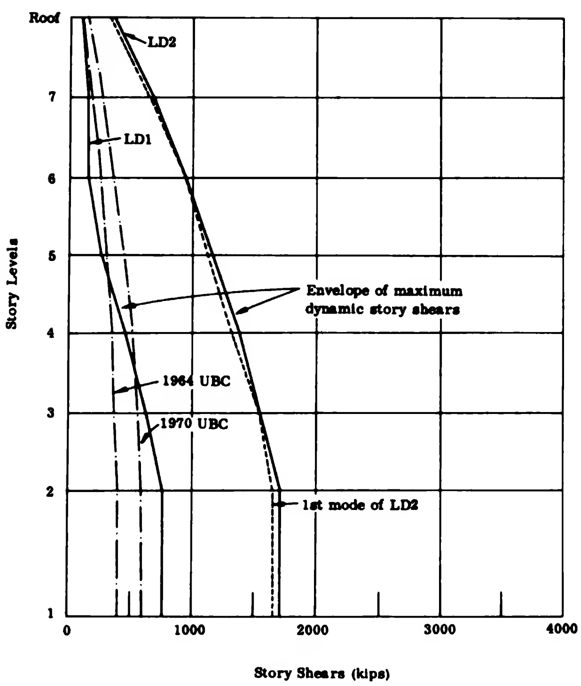

Figure 25c. MAXIMUM STORY SHEARS

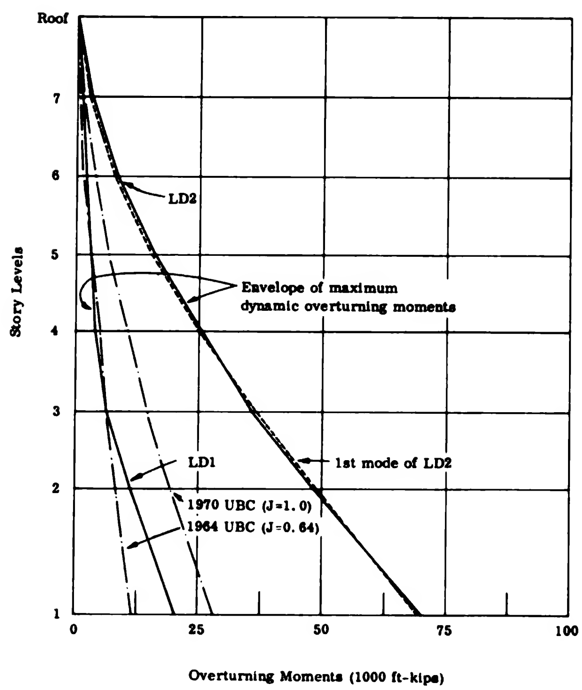

Figure 25d. MAXIMUM OVERTURNING MOMENTS

Figure 25.—Holiday Inn, Marengo Street. Dynamic response and design code values for longitudinal (N38° W.) direction.

Comparisons of girder loads, which include slabs and spandrels, were calculated using ratios of the controlling combinations of vertical load and seismic load moments to estimate ultimate capacities. Table 11 shows these as M/M_u for the transverse direction. Table 12 shows them for the longitudinal direction.

Ultimate moment capacities were computed by the recommendations of reference 3. Generally, girders were under-reinforced, indicating that the cross-sectional area of available steel reinforcement, rather than crushing the concrete, limited ultimate moment capacity. In these calculations, the capacity reduction factor was $\phi = 1.0$.

Tables 11 and 12 divide the results of the girder investigation into two parts. The shorter period models, TD1 and LD1, represent the early portion of the motion. The longer period models, TD2 and LD2, represent the later portion of motion. For the shorter period models, a substantial number of girders have M/M_u ratios greater than 1.0, especially the exterior transverse frames, which have M/M_u ratios equal to or greater than 2.0.

Acknowledging the limitations of these results, there is still a reasonable indication that girders were beginning to yield after about 5 seconds of motion. For the longer period models, the M/M_u ratios are equal to or greater than 1.0 for almost all of the girders shown. Ratios greater than 3.0 were indicated for exterior transverse frames.

The ratio of M/M_u exceeding 1.0 implies that the reinforcing bars are yielding and redistribution of loads is occurring. Therefore, M/M_u values greater than 1 indicate fictitious moments and should be interpreted as such.

Comparison of column loads was made by calculating ratios of the controlling combinations of vertical load and seismic load moments to estimated ultimate moment capacities computed on the basis of the recommendations of reference 3. The corresponding value for calculated axial load, P, is a ratio of P_{uo}. A capacity reduction factor of $\phi = 1.0$ was used for the rectangular tied columns.

Results of the column investigation in tables 13 and 14 are divided into two parts, shorter period models and longer period models. Table 13 gives the results for the transverse frames and table 14, the longitudinal frames.

The ratios of the shorter period models are all well below 1.0, except the corner columns in the transverse direction. About one-half of the moment ratios for the longer period models are equal to or greater than 1.0. The transverse corner columns have ratios greater than 2.0. Acknowledging the above limitations and the indications that girders yielded, it is doubtful that the corner columns experienced some yielding.

Shear capacities of both girders and columns also were checked under the requirements of reference 3. In general, ultimate girder shear capacities were not exceeded by combined vertical and seismic loads. Column shear stresses generally were less than ultimate capacities. However, the interior columns of the transverse exterior frames had calculated shears as high as twice the ultimate capacity. Again, we must acknowledge the limitations of the analysis and the redistribution of loads due to yielding.

DISCUSSION AND INTERPRETATION OF RESULTS

Comparison of Calculated Versus Code Forces

The previous paragraphs presented results of the dynamic analysis and subsequent comparisons of those results with code values. The results of the dynamic analysis, in general, showed that the level of code seismic forces was substantially less than what the structure was required to resist during the earthquake. Maximum base shears were calculated to be four to five times code requirements. Maximum overturning moments at the base of the structure were about nine times greater than code requirements in the transverse direction and about six times greater in the longitudinal direction.

Using the design seismic forces and fundamental period to compute displacement, the design code lateral roof displacement would be approximately 0.03 foot (roughly 0.07*g* at 0.70 second). This indicates that the peak lateral roof displacement was roughly 12 times the code value in the transverse direction and eight times the code value in the longitudinal direction.

Examination of the computed forces in the moment-resisting frames indicated that a majority of the girders experienced excursions beyond their elastic bending moment yield capacities. Except for the exterior transverse frames, the columns remained within their elastic limit capacities. The columns in the exterior transverse frames were computed to have experienced high moment and shear forces. Be-

cause of the apparent yielding that occurred in the girders, these forces may have been redistributed. Uniformly distributed shear forces resulted in peak shear stresses of under 200 psi, within the ultimate shear capacity of the reinforced concrete columns.

Modal Analysis Procedures

In order to test the accuracy of the mathematical models, an attempt was made to verify both mode shapes and periods. Mode shapes could be partially confirmed by the recorded motion by comparing fourth-floor and roof response data; but data would have to be obtained at additional floor-level locations to provide a sufficient number of data points to verify calculated mode shapes.

Period data played an important role in the comparison of the calculated periods with the recorded building periods. This comparison helped immensely to determine the participation of nonstructural elements and the effects of yielding. Since any one of several parameters could be changed to improve the correlation between calculated and measured periods, calculating the correct period did not insure absolutely that the mathematical model truly represented the actual structure. Verification of computed mode shapes by data obtained from building motion records at several intermediate levels would have improved the confidence level of the mathematical models.

Comparison of Recorded and Computed Responses

Comparing acceleration-time histories for the roof and fourth-floor levels yielded comparisons of recorded and computed responses. The general shape of the computed time histories for particular time intervals correlates reasonably accurately with the recorded values (figs. 22 and 23).

Calculated fundamental periods for the structure (based on a bare structural frame) were approximately 0.79 second in the longitudinal direction and 0.88 second in the transverse direction. The apparent recorded fundamental period of the structure during the first 5 seconds of the earthquake was roughly 0.60 second in the longitudinal direction and 0.63 second in the transverse direction. The apparent recorded period of the structure during the latter portion of the earthquake was roughly 1.10 seconds in the longitudinal direction and 1.15 seconds in the transverse direction.

The shorter fundamental periods for the first few seconds of motion suggested that architectural elements initially provided the structure with additional horizontal rigidity. Guest rooms have a considerable number of party walls. There are exterior plaster walls and brick walls at the first floor. Although these walls are not considered part of the lateral force-resisting system, they do resist lateral forces, especially during the first part of the earthquake.

After 5 seconds of motion, the period of the structure appeared to lengthen in both directions. This indicated that after this time, enough force had been generated to overcome most or all of the resistance of the nonstructural elements. In addition, the girders of the bare structural frames began to yield to further lengthen the periods.

Tables 8 and 9 list computed and recorded maximum accelerations and displacements. Comparisons between the computed and recorded results show reasonably good agreement.

Correlation With Damage Observations

The analyses have indicated that beams and slabs yielded, that columns generally remained within elastic limits (but the columns of transverse exterior frames were highly stressed), and that moderately large interstory displacements, greater than 0.05 foot or ½ inch, occurred.

Observed damage included signs of yielding beams (figs. 8 through 12) and substantial nonstructural damage. No serious column damage was observed.

Beam damage appears less severe than one might imagine from a study of the results of the analysis. Yet these observations appear reasonable when compared to observations made on concrete test structures located at the Nevada Test Site (references 15 and 16). Observed partition damage also appears consistent when compared to published data (reference 11).

Ductility

The relatively large relative displacements exhibited by the analysis indicated a high degree of inherent ductility in the structure. A rough approximation of an average ductility ratio can be found by

comparing the peak calculated fundamental mode relative lateral roof displacement with the equivalent displacement at the nominal elastic limit. The elastic limit displacement has been calculated to be 0.08 foot. This is roughly 2½ times the design code displacement.

Using the peak first-mode displacements shown in tables 8 and 9, the approximate ductility in the transverse direction would be 4. The approximate ductility in the longitudinal direction would be 3. The analysis of member forces indicates that this ductility was exhibited in the beams and slabs while the columns essentially remain elastic.

Additional Period Data

In addition to the accelerograph recordings taken during the earthquake, building period observations were made before and after the earthquake. (See "Building Period Measurements Before, During, and After the San Fernando Earthquake.")

In the referenced observations, measured fundamental building periods (0.5 to 0.6 second) were significantly shorter than the periods observed during the earthquake; but postobservation periods were longer than preobservation periods. During these observations, the amplitude of motion was less than 1 percent of the recorded earthquake motion.

The results of these observations emphasize the greater participation of nonstructural elements at small amplitudes of motion. The relationship of the pre- and postearthquake observations to the earthquake observations agrees with similar types of observations in references 15 and 16.

SUMMARY OF FINDINGS

The dynamic analysis of the Holiday Inn on Marengo Street yielded the following conclusions about the effects of the San Fernando earthquake.

1 During the earthquake, the structure responded at amplitudes that exceeded the elastic limits of a substantial number of girders.

2 Calculated earthquake forces exceeded prescribed code minimums by a factor of 4 or 5.

3 Interstory displacements exceeded ½ inch. This accounts for the moderate amount of nonstructural damage.

4 On the basis of calculated building response, maximum lateral displacements were from eight to 12 times prescribed code minimums, but observed structural damage was relatively minor.

5 Estimates of ductility indicate average ratios of 4 for beams and slabs in the transverse direction and 3 for beams and slabs in the longitudinal direction.

6 Damping was approximated to be 2 to 5 percent of critical viscous damping.

COMPARISON OF ORION AVENUE AND MARENGO STREET STRUCTURES

The investigation of these two structures has been fairly extensive. Some comparative conclusions regarding these two structures follow.

1 The structures are essentially identical. The only physical differences are the locations of the buildings, orientations of the structures, orientations of the elevators, and locations of some "Holiday Inn" signs.

2 The elastic limits of structural elements of both structures were exceeded during the earthquake.

3 Although the peak accelerations of both structures were roughly equal, the Orion Avenue structure, which was closer to the epicenter, experienced almost twice the peak lateral displacement of the Marengo Street structure.

4 The Orion Avenue structure exhibited a greater loss of stiffness (greater increase of period) during the earthquake than the Marengo Street structure.

5 The cost of damage repair for the Orion Avenue structure was roughly 1.5 times the cost of damage repair for the Marengo Street structure. However, visual observations indicate that the actual severity of the damage was greater than that ratio.

RECOMMENDATIONS

This study recommends several areas of possible change in seismic design and instrumentation.

Dynamic Analysis

Presently, the state-of-the-art of structural engineering includes some very useful methods for the dynamic analysis of structures. These include time-

history analysis, response spectra model analysis, and computerized analytical methods of structural analysis. With the growing use of these complex analytical methods, design codes should establish criteria to govern the use of these procedures. The results of the analysis of the Holiday Inn structure have illustrated the variation of results that can be obtained because of changes in natural periods of vibration. It should be noted that none of these methods provide exact solutions to dynamic response. Considering their limitations, they yield reasonable representations. Some of the limitations include instrument accuracy, time scales, phase relationships between modes in both recorded and calculated time histories, random variables, human error, and effects of instrument locations.

Modal Periods

These reports illustrate the nonlinearity of structural response and its effects on modal periods. Similar findings have been observed in other investigations. Most dynamic analysis procedures consider each period to have a constant value. Yet it appears that structures should be analyzed for a spectrum of periods. The value for each period would be dependent on amplitude of building motion. Design criteria should be established to incorporate these effects of potential period variation.

Nonstructural Elements

As has been illustrated in these reports, the partitions as well as other nonstructural elements can play an appreciable role in the character of the structural response of a building to an earthquake. The effects of nonstructural elements can add to or subtract from the lateral capacity of structures. Therefore, it is recommended that the effect of nonstructural elements be included in the lateral force design criteria.

Strong-Motion Instrumentation

Strong-motion recording devices should be placed to provide a record of true transverse and longitudinal motion without any undue contributions from real or accidental torsional effects. Vertical records should be taken at or near a column to avoid local, possibly anomalous effects from more flexible elements such as thin slabs. It also may be desirable to locate some vertical recorders at the center of typical floor elements to measure possible response amplification.

Verification of mode shapes could be improved if a greater number of recording instruments were placed between the top and ground levels of highrise buildings. By placing only one intermediate or midheight instrument, verification may be doubtful.

Bunker Hill Tower (31)

800 West First Street, Los Angeles

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San Francisco, Calif.

DESCRIPTION OF BUILDING

Bunker Hill Tower, a 32-story steel frame structure in downtown Los Angeles, is situated some 26 miles south of the epicenter in an area generally called the Los Angeles Basin.

Completed in 1968 at a cost of approximately $7 million, the building (figs. 1, 2, and 3) is an office and apartment structure. Plan dimensions are 90 feet wide by 125 feet long for all 32 stories. Story heights are typically 10 feet 2 inches, except at the first four stories and the story below the main roof where slightly larger heights were used. The total height of the structure from the ground floor to the main roof is 337 feet 6 inches. A mechanical penthouse covers approximately 30 percent of the main roof area.

An adjoining structure, consisting of a plaza and three underground parking levels, is located on the north side of the tower, but is completely separated from the tower at all levels by seismic separation joints. The analyses and results discussed in this building report are limited to the 32-story tower portion.

Underlying soils at the site consist primarily of firm shale and sandstone. The upper shale is fractured and weathered, and the deeper shale is less weathered. Bedding in the upper light brown shale is downward to the south at an angle of from 40° to 60° below the horizontal, but is indistinct in the dark grey shale below. A typical soil boring log is shown in figure 4 (reference 18).

Foundations for the tower consist either of concrete spread footings, continuous strip footings, or grade beams on caissons (fig. 5). Interior columns, which are not part of the lateral force-resisting system, are supported on individual spread footings. Perimeter columns, which support the lateral force-resisting system, are supported by a continuous foundation. A conventional, continuous spread footing is used on the northern half of the building, and a con-

Figure 1.—Bunker Hill Tower. North elevation. Julius Shulman photograph.

Figure 3.—Bunker Hill Tower. Last of the prefabricated sections of exterior frame being lifted into place. Tech Photo photograph.

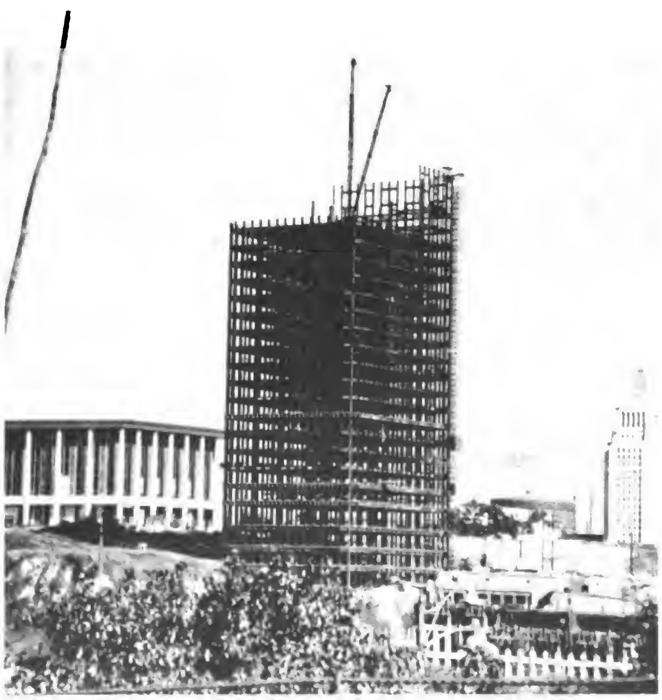

Figure 2.—Bunker Hill Tower. Southwest elevation of steel frame during construction. Tech Photo photograph.

tinuous grade beam supported on belled caissons is used on the southern half of the building. The caisson system was used to extend part of the foundation to deeper strata to avoid imposing stress on an existing tunnel located near the south side of the tower.

Design bearing pressures for dead and live loads were 11 ksf (kips per square foot); for lateral forces, a coefficient of friction of 0.4 or a passive resistance taken to be 1 ksf at the surface and increasing at a rate of 0.3 ksf per foot of depth was allowed (reference 18).

Lateral forces in each direction are resisted by tube action of the perimeter moment-resisting space frames consisting of girders and columns (figs. 6 and 7). Interior girders and columns are designed to carry only vertical loads. All exterior girder-to-column connections and column splices are butt welded to develop the full moment capacity of the members (fig. 8). Where required, web doubler plates and horizontal diaphragm plates were used in the girder-column connection to reinforce the panel zone against a shear failure in the column webs. All mem-

bers were fabricated from rolled shapes, except the corner columns which are box sections of fabricated plate. A list of properties of the materials used in construction is given in table 1.

The structure was designed in 1967 under the requirements of the Los Angeles City Building Code. Design live loads were 100 psf for the lower three (office) floors and 40 psf for the upper (apartment) floors. Typical floor construction consists of a 5-inch lightweight concrete slab on steel beams and girders. All beams and girders were fireproofed with spray-on fireproofing, and all columns were fireproofed with gypsum wallboard. Permanent nonstructural partitions consisting of gypsum wallboard on metal studs enclose the elevator shafts, stairwells, duct shafts, apartments, and toilets. Building enclosure around the perimeter typically consists of glass window walls between columns. Concrete block walls are used at the lower floors below the plaza level, with the concrete block walls detailed to allow the steel frame to move independently of the block walls.

According to the designer, standard inspection procedures were followed during construction, and a full-time licensed inspector was present to inspect the construction and interpret the design drawings.

EARTHQUAKE DAMAGE

The owner of the building reported that no earthquake damage to any structural elements was observed, and that only minimal damage to nonstructural elements occurred. Nonstructural damage occurred as cracking to drywalls and ceilings, and only minor patching and painting were required to restore the damaged areas. The building owner also reported that three windows were broken by objects which fell against the windows during the earthquake.

Four elevators were temporarily out of service, two for approximately 2 hours, one for 1 day, and another for 2 days. A cable had to be replaced on one of the elevators when a kink formed after the cable had jumped its sheave. Damage repair costs were unavailable.

RECORDED EARTHQUAKE RESPONSE

Earthquake motion in the Bunker Hill Tower was recorded by three New Zealand MO–2 strong-motion accelerographs. Instruments were located at the roof (33d floor), 16th floor, and ground (first) floor.

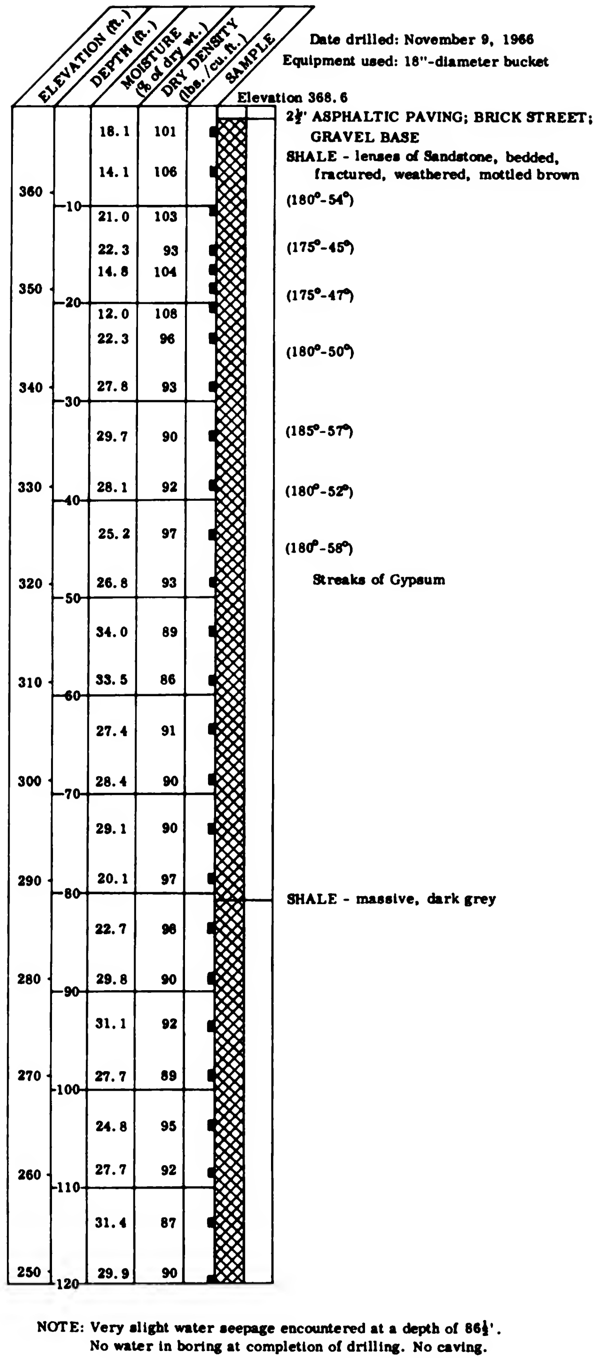

Figure 4.—Bunker Hill Tower. Log of typical soil boring.

The MO–2 instrument utilizes an optical recording system and photographic film strip to record the longitudinal, transverse, and vertical components of acceleration. Acceleration time-history plots, obtained from digitized listings of the photographic records of motion, are shown in figures 9, 10, and 11. Peak recorded accelerations, also from the digitized records, are summarized in table 2.

Table 1.—Properties of construction materials

Concrete

Location in structure	Aggregates	Unit weight	Minimum specified compressive strength (f'_c)	Modulus of elasticity (E)
		pcf	*psi*	*psi*
Above first floor.	Lightweight (ASTM C-330)	110	3,000	2.1×10^6
First floor and below.	Regular weight (ASTM C-33)	150	3,000	3.3×10^6

Reinforcing Steel

Location in structure	Grade	Minimum specified yield strength (f_y)	Modulus of elasticity (E)
		psi	*psi*
Typical all locations.	Intermediate (ASTM A-615, Grade 40)	40,000	29×10^6

Structural Steel

Location in structure	Grade	Minimum specified yield strength (f_y)	Modulus of elasticity (E)
		psi	*psi*
Box columns, below 5th floor.	Structural steel (ASTM A-441)	46,000	29×10^6
Typical except as noted.	Structural steel (ASTM A-36)	36,000	29×10^6

The digitized acceleration records used in the analytical portion of this building report are for the first 40 seconds of recorded motion. After most of the analytical work had been completed, longer records of about 60 seconds' duration became available, but unfortunately could not be used.

From digitized versions of recorded ground-level motion, response spectra were determined for 1, 5, and 10 percent of critical damping. These spectra are presented in figures 12 and 13 for the transverse and longitudinal components of motion, respectively. The spectra have been shown on four-way log paper to facilitate the reading of pseudo-absolute acceleration, pseudo-relative velocity, and relative displacement values.

During the preparation stages of this report, it was discovered that there had been some digitization errors in the strong-motion records (figs. 9, 10, and 11) used in the analyses presented herein. The digitization of the time scale from the photographic filmstrip was based on an assumed average film speed of 1.5 cm/sec. A review of the original records indicates, however, that the average film speed of the instruments at the roof and 16th levels was greater than the assumed value. The situation was complicated further by the fact that no time marks were visible on the first-floor records. Because very little time was available to fully examine the possible implications

Table 2.—Peak recorded accelerations

Station	Transverse (N.37°E.) component	Longitudinal (N.53°W.) component	Vertical component
	g	*g*	*g*
Roof (33d) floor...	0.186	0.294	0.224
16th floor.........	.121	.182	.158
1st floor (ground)...	.094	.143	.059

of these digitization errors, let alone to redigitize the film records and perform additional computations, no correction of the digitization errors has been made. However, the following discussion, based on a brief study of the implication of these errors, is offered.

A review of the original records by the California Institute of Technology (conducted at the request of the Blume firm) showed average film speeds of 1.76 cm/sec for the roof level and 1.62 cm/sec for the 16th floor. Since the results of this building report, based on the uncorrected digitized records, show a good correlation between computed and recorded responses at roof level, it is felt that: (1) The film speed for the ground-level records is about the same as that for the roof-level records; and (2) the natural periods of the structure actually are slightly smaller (by proportionately the same amount as the indicated change in time scale of the digitized records) than the values reported herein.

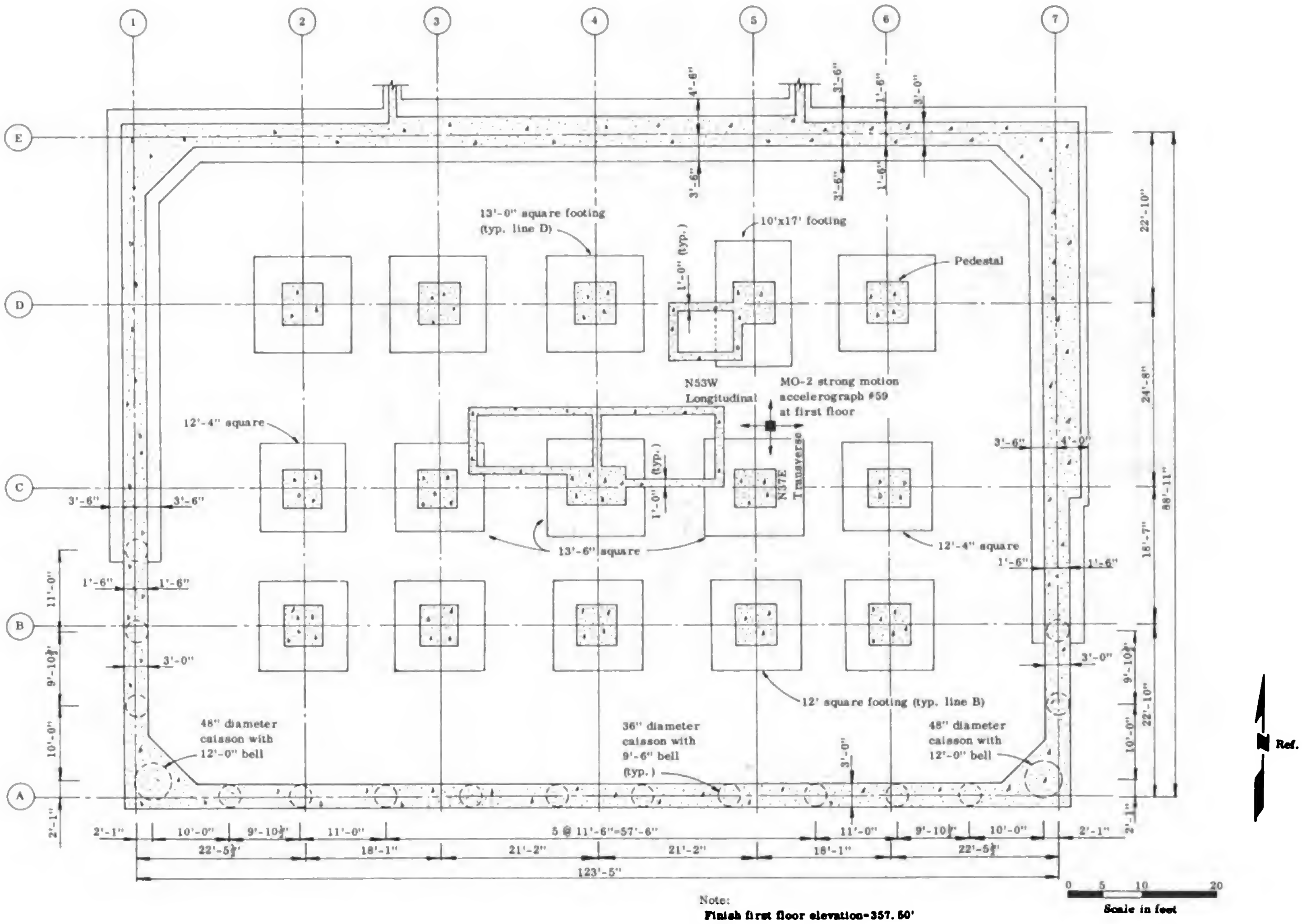

Figure 5.—Bunker Hill Tower. Foundation plan.

MATHEMATICAL MODELING

As discussed in the Description of Analytical Procedures section, mathematical models were developed to model the physical characteristics of the tower in its response to earthquake ground motion. Two models were developed, one for each principal horizontal direction. Each model assumed the member stiffness properties indicated in table 3. Values of the gross moment of inertia (I_o) of the steel girders were adjusted to compensate for the composite action of the 5-inch floor slab (fig. 8) and, to a lesser extent, the spray-on fireproofing. Column properties were not adjusted.

Table 3.—Member stiffness properties

Member	Moment of inertia	Shear area	Cross sectional area
Girders	$2I_o$	....	....
Columns	I_o	A_{web}	A_g

Several other adjustments in modeling the tower were required. Because of the close column spacing and rigidity of the girders, tube action occurs in the lateral force-resisting frames (fig. 14). To approximate tube action effects the procedure indicated in reference 19 was employed. This procedure, which allows the use of planar analysis programs, is considered to provide good approximations of the three-di-

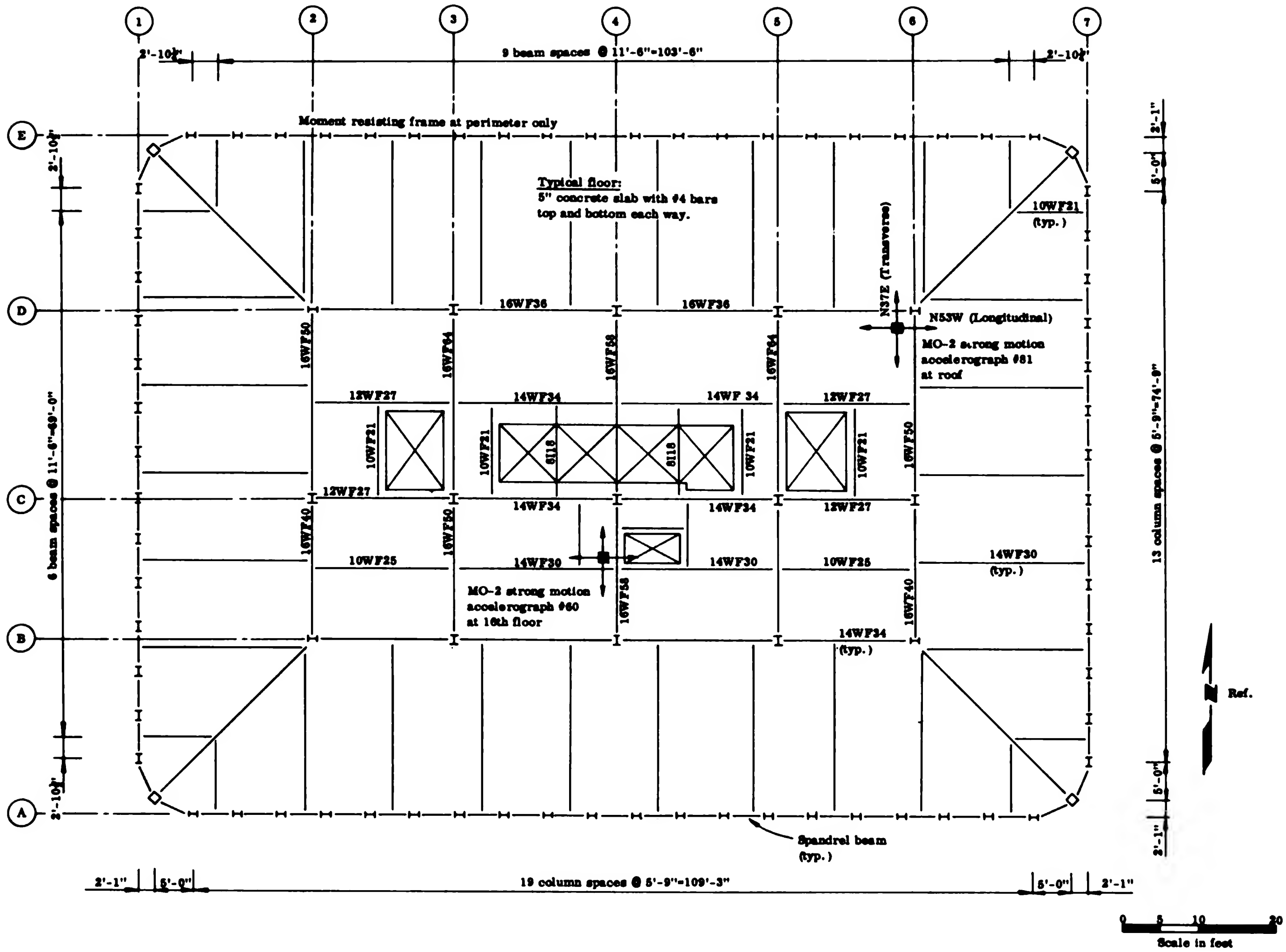

Figure 6.—Bunker Hill Tower. Typical floor framing plan.

mensional tube action effects. The analysis assumed that no eccentricity existed between center of mass and center of rigidity. This assumption is considered reasonable in view of the excellent symmetry found in the structure.

At each story, the lightweight concrete floor slab was assumed to act as a rigid horizontal diaphragm. The largest width-to-depth ratio of the diaphragm is 1.4, indicating a very rigid diaphragm. The openings that occur in the diaphragm are near its center, but these are not expected to reduce rigidity significantly.

Several values of damping were used in the analyses. The values indicated in table 4 are those values which gave best correlation with recorded response.

RESULTS OF ANALYSIS

The mathematical models described in the previous section were developed from a careful assessment of what the actual dynamic properties of the structure would be. Only damping—and not member stiffness properties—was adjusted for best correlation. As is common in current dynamic analysis procedures, damping was kept constant for all modes and was not varied on a mode-by-mode basis. In the transverse direction, 3-percent damping, instead of the more usual 2 percent, gave reasonable correlation with recorded acceleration amplitudes, while 1-percent damping was used in the longitudinal direction.

Table 4.—Mathematical models used in the analysis

Building direction	Fundamental period	Purpose	Earthquake time interval	Number of modes	Applied viscous damping
	Seconds		*Seconds*		*Percent*
Transverse	3.98	Mode shapes, periods, and dynamic analysis.	0 to 39.6	6	3
Longitudinal	3.51	..........do..........	0 to 39.6	6	1

Mode Shapes and Periods of Vibration

For both principal directions, mode shapes and periods of vibration were calculated for the first six translational modes, and all six modes were used in the analysis. Mode shapes and periods for the first three modes are shown in figure 15.

Computed Floor Accelerations

As a major part of this building report, correlation studies of recorded and computed floor accelerations were made for the roof level. Because of the difficulties in digitization previously described, no comparison was made for 16th-floor response. Results of these studies are shown in figure 16. Table 5 provides a comparison of recorded and computed peak floor accelerations and computed maximum interstory drifts and total displacements.

Table 5.—Maximum accelerations and displacements at roof and 16th floor

Direction of response	Maximum parameter	Roof	16th floor
Transverse	Computed acceleration (g)	0.215	0.098
	Recorded acceleration (g) [1]	.188	.130
	Computed displacement (in.)	20.34	8.50
	Computed story drift (in.)	.72	.70
Longitudinal	Computed acceleration (g)	.212	.171
	Recorded acceleration (g) [1]	.293	.186
	Computed displacement (in.)	14.09	6.64
	Computed story drift (in.)	.42	.49

[1] Obtained from listing of digitized records.

Maximum Building Displacements

Envelopes of both maximum total dynamic displacements and corresponding maximum interstory displacements are shown in figures 17a and 18a. These both show the peak displacement of each story with respect to the base and the peak floor-to-floor differential displacement for each direction. Not all of the maximums necessarily occurred simultaneously. In the transverse direction, maximums occurred between 20.8 and 22.6 seconds after the start of motion, while in the longitudinal direction, maximums occurred between 22.0 and 25.4 seconds after the start of motion.

Also plotted in figures 17a and 18a are envelopes of maximum displacements due to design seismic forces. These forces were associated with a design base shear of approximately 0.03W, in contrast to the 0.023W minimum (table 6) specified by the 1970 UBC (reference 4).

Table 6.—UBC seismic design parameters

Code	Z	K	C	Base shear V = ZKCW	Period (T)	J
					Seconds	
1970 UBC	1.0	0.67	0.034	0.023W	3.2	0.45

Maximum Story Forces

Maximum horizontal story forces determined from the dynamic analysis and corresponding minimum code values are plotted in figures 17b and 18b. For each level, dynamic story forces were determined as the product of story mass times maximum absolute story acceleration.

Maximum Story Shears

Maximum story shears were calculated as part of the dynamic analysis. These are indicated in figures 17c and 18c along with corresponding minimum code values. For clarity, shear curves have been shown as smooth, instead of stepped-line, curves. Code design shears have been computed under the requirements of the 1970 UBC. The UBC story shears for both directions are based on the numerical coefficients and period shown in table 6.

Story shears, which were determined from the dynamic analysis indicated in figures 17c and 18c, are peak values calculated to have occurred at each floor level at some time during the earthquake excursion. These peak values, however, did not necessarily occur simultaneously and should be considered to

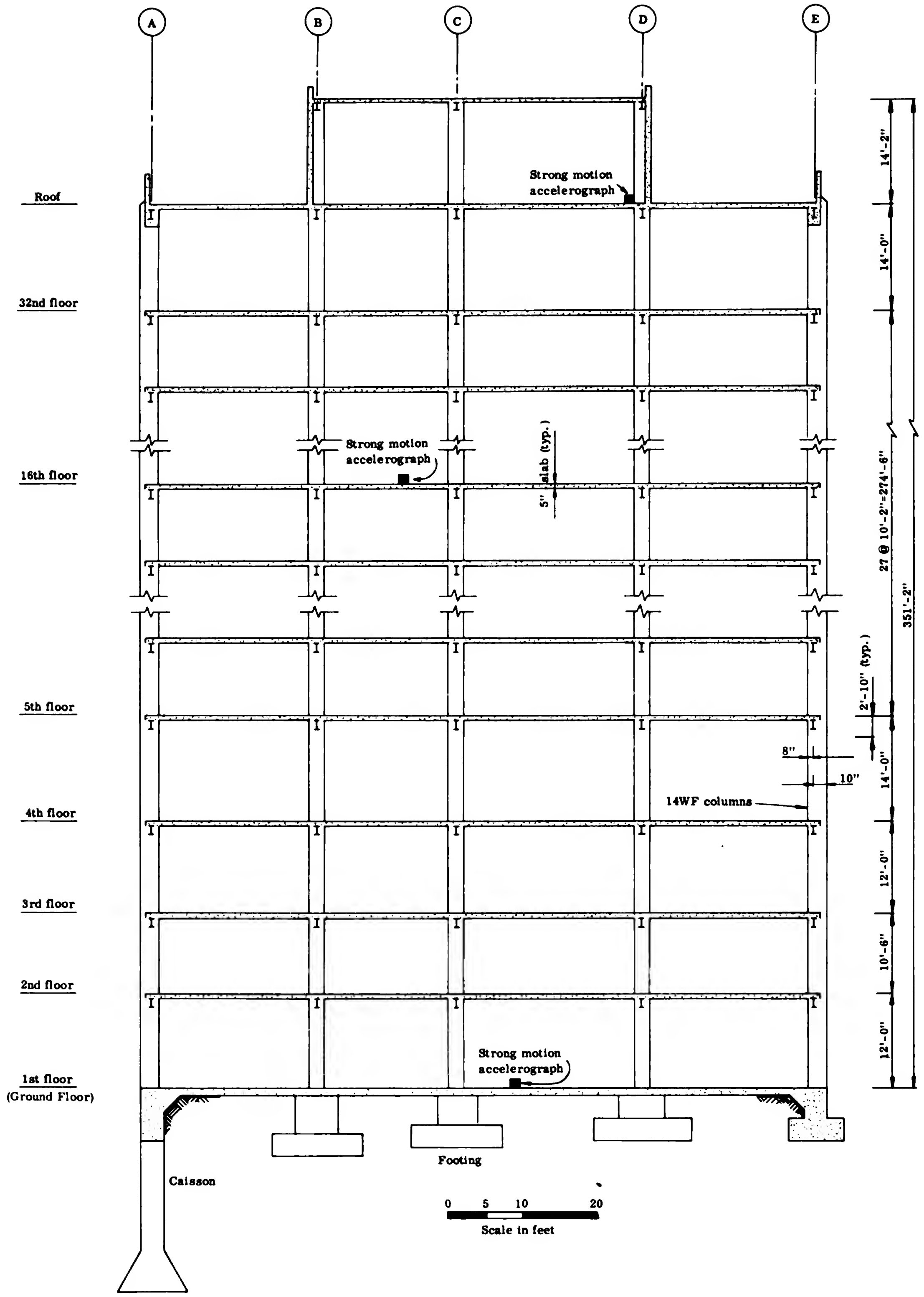

Figure 7.—Bunker Hill Tower. Typical transverse section.

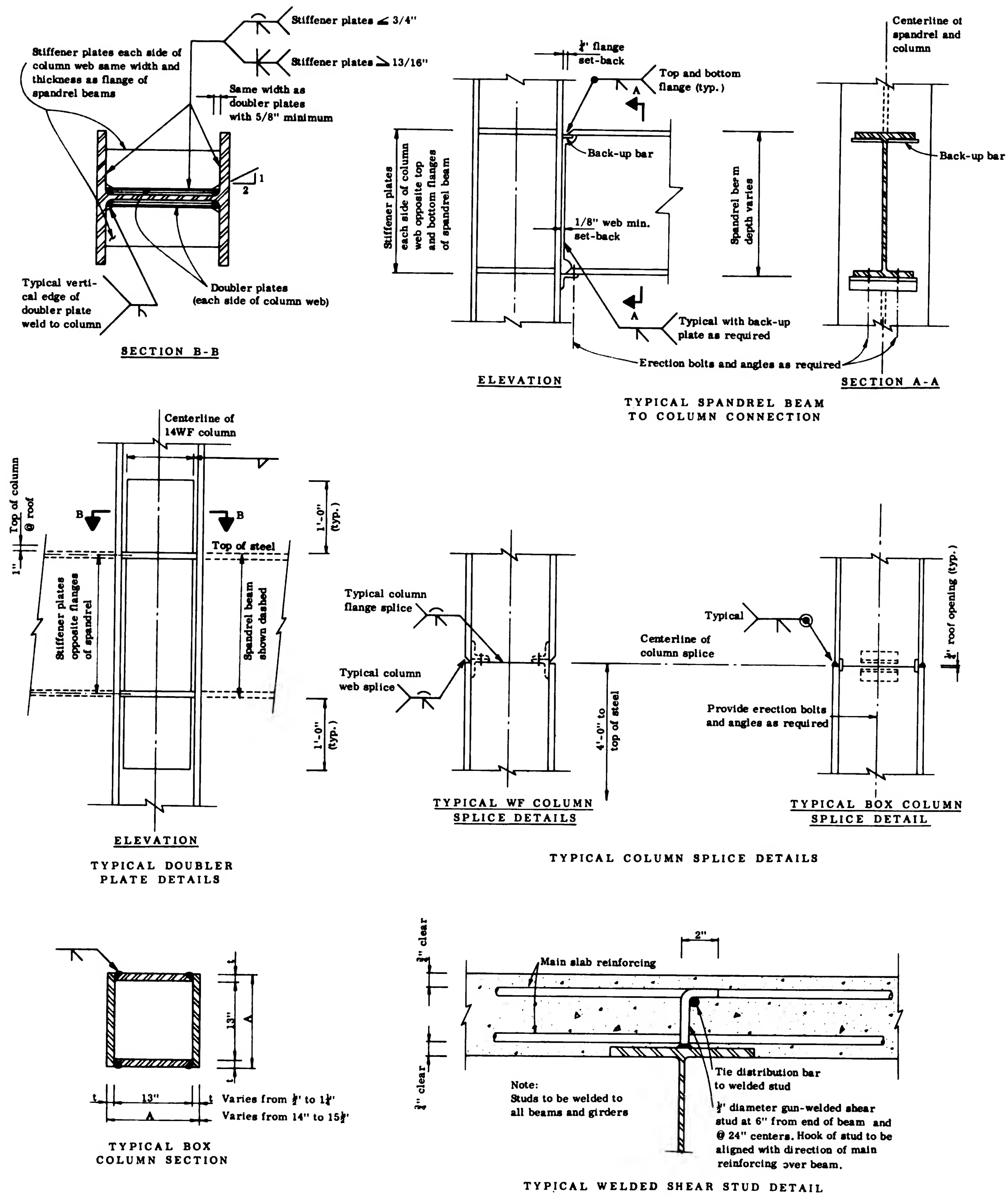

Figure 8.—Bunker Hill Tower. Typical structural details.

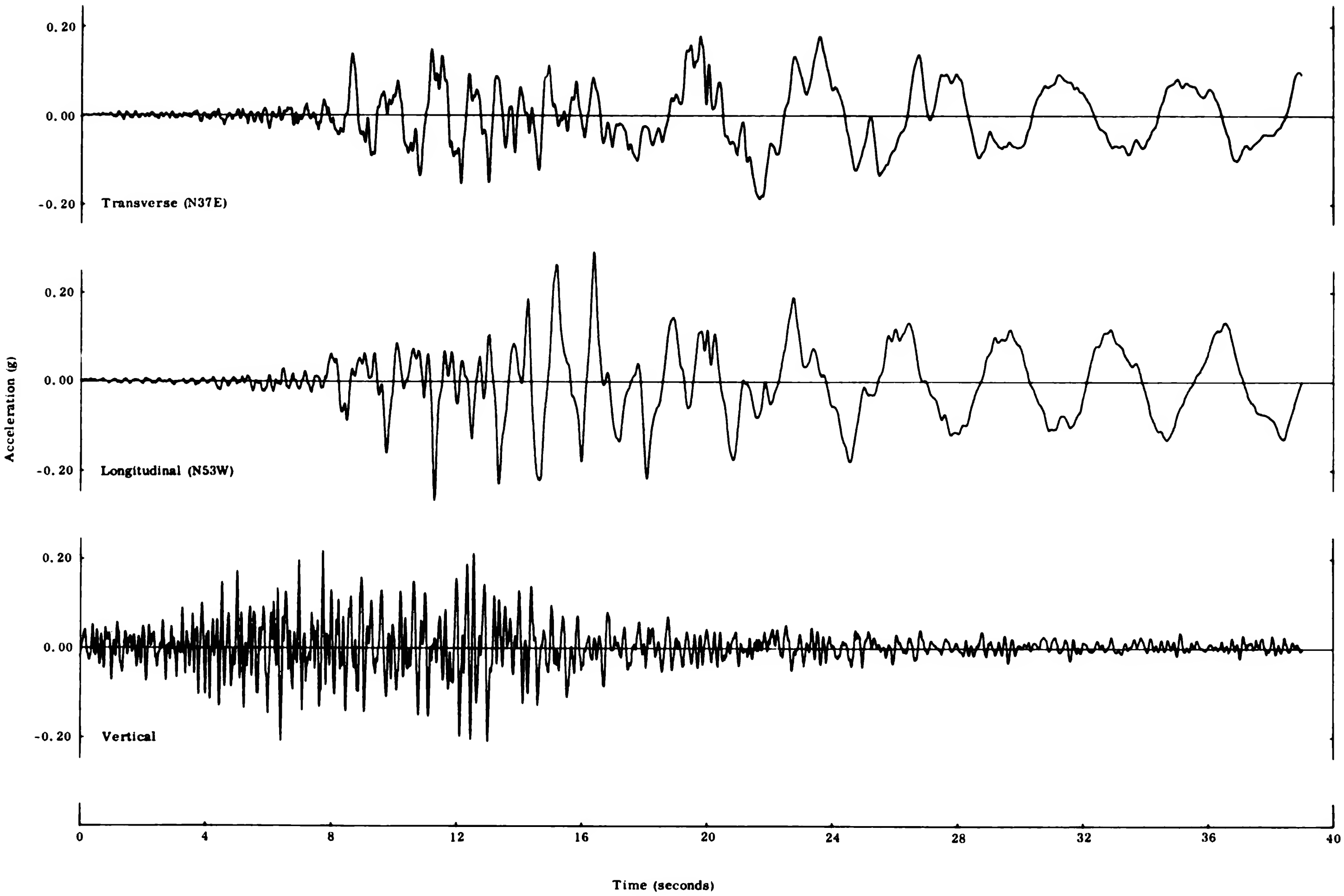

Figure 9.—Bunker Hill Tower. Recorded acceleration at roof.

represent only an envelope of maximums. The story shears indicated are due to the contribution of the first six modes of vibration, but the first-mode contribution accounted for virtually all of the total response. The second- and third-mode values indicated are the contributions of these modes to total response at the time of maximum response.

Maximum Overturning Moments

In a manner similar to that used in determining the maximum story shears, the envelope of maximum overturning moments was calculated. Calculated overturning moment values are compared to those determined under various minimum code requirements in figures 17d and 18d. As in the case of story shears, the first six modes of vibration were used, but the first mode contributed almost all of the total response. Second- and third-mode contributions at the time of maximum response are also shown. These are relatively small, however.

Comparison of overturning moments calculated in the dynamic analysis with those calculated under minimum requirements of the 1970 UBC with $J = 0.45$ and $J = 1.0$ also has been indicated in figures 17d and 18d. After publication of the 1970 UBC, the J factor for determining the base overturning moment was the subject of considerable scrutiny. Table 6 indicates that the 1970 UBC would require a minimum J value of 0.45 for both the transverse and longitudinal directions, but subsequent amendments to the 1970 edition of the UBC have eliminated the J-factor concept, thus, in effect, increasing the minimum J value to 1.0. For this reason, overturning moments determined with a value of $J = 1.0$ also have been included as a comparison with the 1970 UBC prescribed minimums.

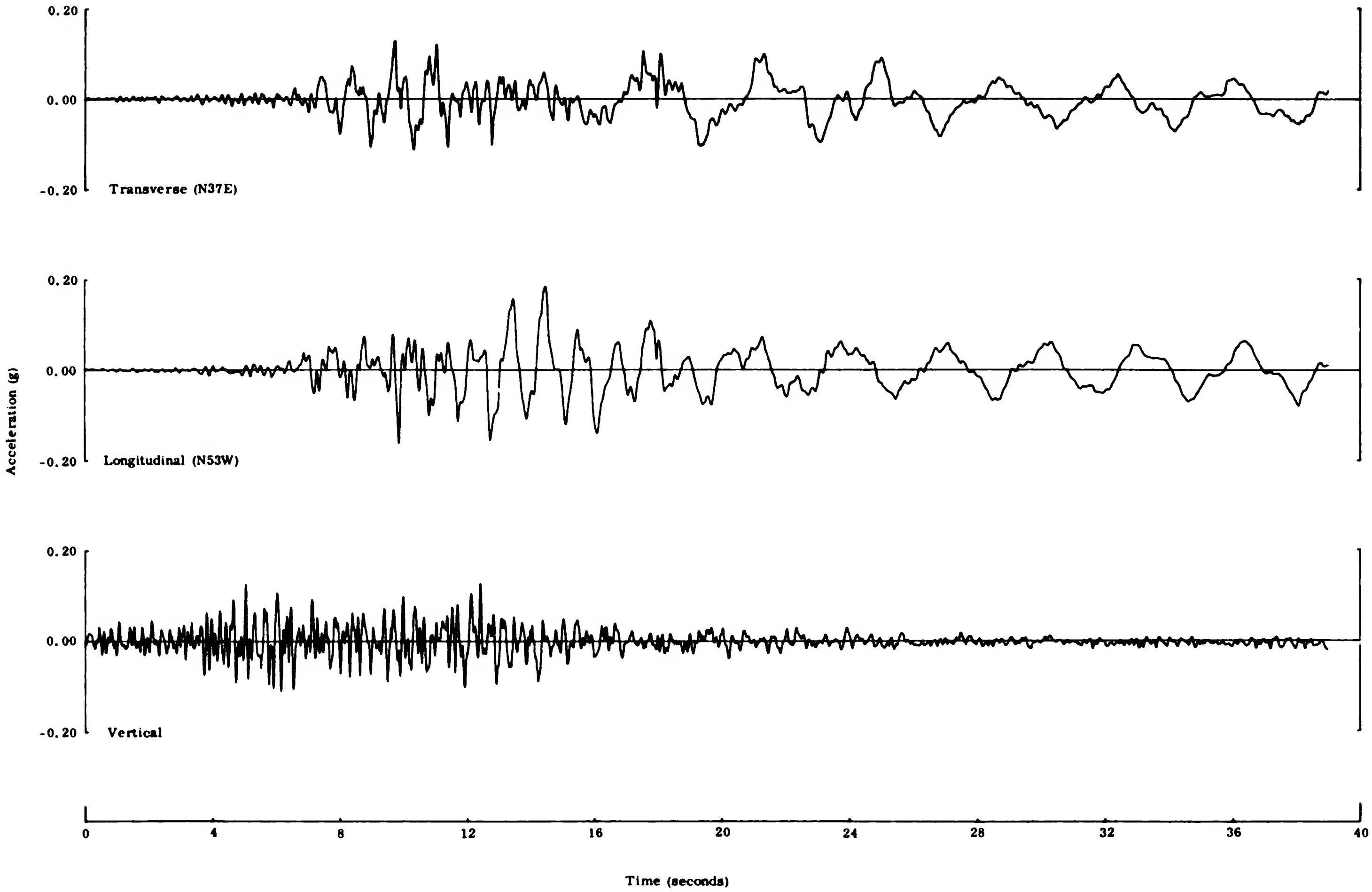

Figure 10.—Bunker Hill Tower. Recorded acceleration at 16th floor.

Loads on Key Girders

Earthquake loads on girders of the lateral force-resisting frames were investigated, first by computing seismic plus estimated actual vertical loads and then by comparing these values to the estimated ultimate member capacity. Specifically, comparisons were made by calculating ratios of the controlling combinations of vertical and seismic moments (M) to calculated plastic moment capacities (M_p). These are indicated as M/M_p in table 7. Only the maximum value of M/M_p that occurred in any of the girders on a given floor is shown in table 7.

Table 7.—Summary of girder $\frac{M}{M_p}$

Floor level	Transverse frames	Longitudinal frames
Roof	0.34	0.29
30th	.86	.51
28th	.88	.60
26th	.90	.66
24th	.82	.70
22d	1.01	.87
20th	.71	.74
18th	.75	.82
16th	.64	.58
14th	.62	.55
12th	.58	.53
10th	.57	.54
8th	.64	.58
6th	.59	.56
4th	.68	.73
2d	.75	.90

Plastic moment capacities were computed on the basis of the recommendations of reference 2. The degree to which a member is loaded is indicated by the M/M_p ratio. Values less than 1.0 indicate that significant girder yielding did not take place. In all calculations, member end moments were used and plastic moment capacities were determined using the plastic section modulus, Z_p.

Shear capacities of the girder members were also checked under the requirements of reference 2. In

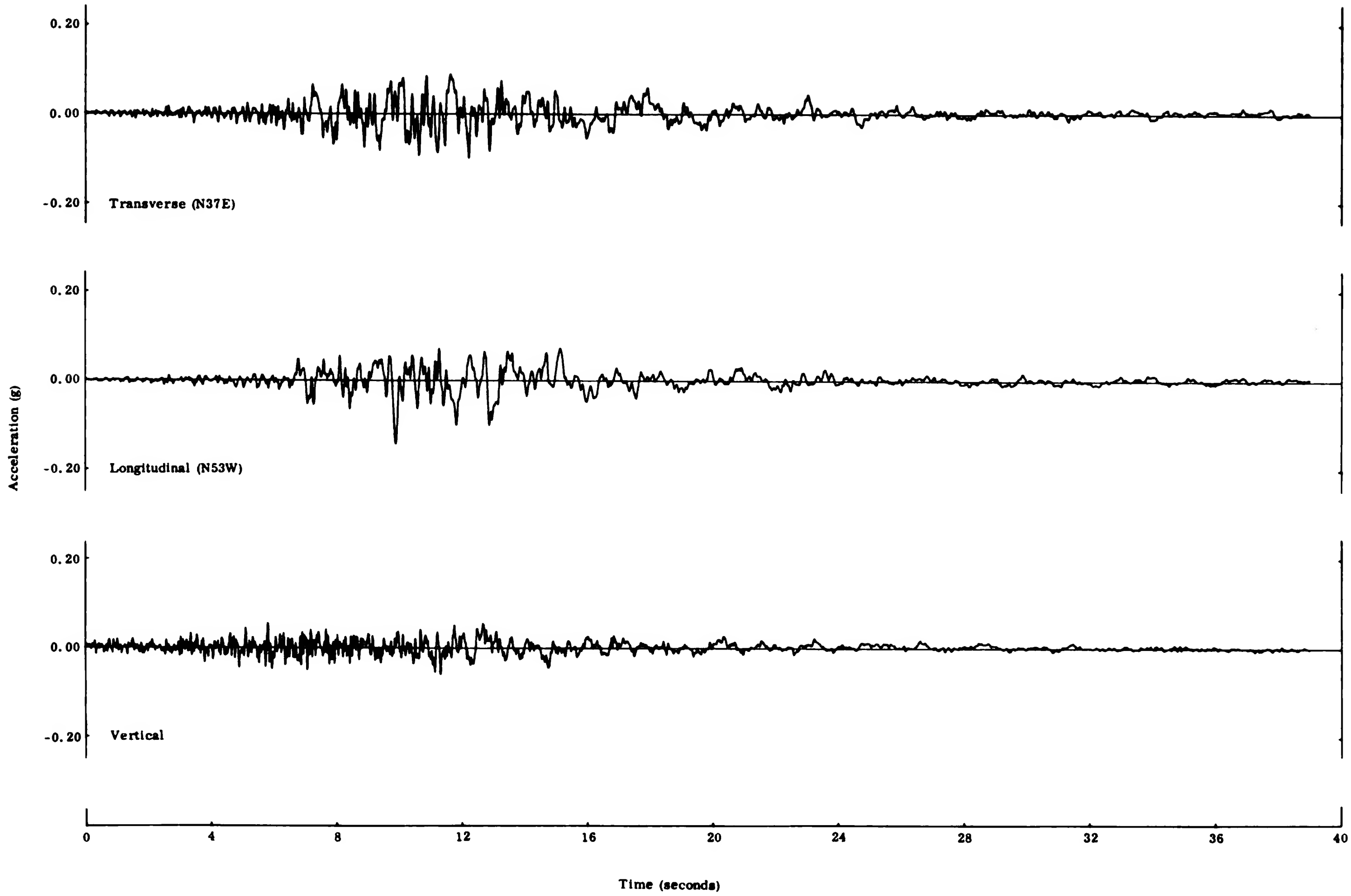

Figure 11.—Bunker Hill Tower. Recorded acceleration at first floor.

general, ultimate shear capacities were not exceeded by combined vertical and calculated earthquake loads.

Loads on Key Columns

The effects of calculated earthquake loads on columns of the lateral force-resisting frames also were studied. The results of these studies are briefly summarized in tables 8 and 9. Ratios of controlling combinations of vertical and seismic moments (M) to plastic moment capacity (M_{pc}) have been calculated for key columns of each frame. Moments (M) include moments from both estimated actual vertical loads and moments determined from the dynamic analysis. Values of M/M_{pc} greater than 1 indicate that yield conditions have been exceeded. Values smaller than 1 indicate that yielding has not occurred. Estimates of modified plastic moment capacities (M_{pc}) were computed on the basis of the recommendations of reference 2, considering the moment capacity reducing effects of simultaneous vertical and seismic axial forces.

Response to Different Ground Motions

The effects of three different earthquake ground motions on the Bunker Hill Tower were studied to compare the response of the tower had it been located on different sites and experienced other earthquake motion. Comparisons were made of story shears and overturning moments determined by dynamic analysis methods and similar values computed for the minimum requirements of the 1970 UBC with J = 1.0. The study was limited to responses in the transverse direction (fig. 19).

The three earthquake ground motions used in these comparisons are:

1. 1940 El Centro earthquake, north-south component

Table 8.—Summary of column interaction $\frac{M}{M_{pc}}$ for transverse frames

Floor level	Corner column	1st column from corner	2d column from corner	Typical columns
32d	0.20	0.26	0.34	0.45
29th	.31	.48	.60	.78
27th	.42	.56	.70	.87
25th	.43	.48	.62	.80
23d	.55	.60	.77	.90
21st	.69	.62	.79	.90
19th	.62	.60	.76	.89
17th	.79	.63	.78	.90
15th	.86	.62	.74	.83
13th	.96	.59	.69	.76
11th	.86	.62	.73	.77
9th	.94	.59	.67	.73
7th	.83	.65	.74	.76
5th	.84	.65	.70	.72
3d	.83	.85	.89	.81
1st	1.10	.76	.71	.71

NOTE.—The following approximate interaction equations were used to determine M_{pc}:

1. For strong axis bending of WF column sections

$$M_{pc} = 1.18\left(1.0 - \frac{P}{P_y}\right)M_p \quad \text{where } \frac{P}{P_y} > 0.15$$

$$M_{pc} = M_p \quad \text{where } \frac{P}{P_y} \leq 0.15.$$

2. For biaxial bending of symmetrical box section

$$M_{pc} = \left(1.0 - \frac{P}{P_y}\right)M_p.$$

Table 9.—Summary of column interaction $\frac{M}{M_{pc}}$ for longitudinal frames

Floor level	Corner column	1st column from corner	2d column from corner	Typical columns
32d	0.14	0.21	0.24	0.31
29th	.17	.30	.36	.50
27th	.26	.40	.48	.63
25th	.31	.43	.53	.68
23d	.34	.47	.57	.71
21st	.46	.58	.70	.82
19th	.41	.51	.62	.74
17th	.52	.56	.67	.80
15th	.51	.50	.60	.70
13th	.54	.47	.56	.66
11th	.46	.49	.59	.66
9th	.51	.51	.60	.66
7th	.66	.50	.61	.66
5th	.50	.51	.58	.60
3d	.54	.60	.62	.62
1st	.79	.61	.61	.63

NOTE.—See footnotes of table 8.

2. 1971 San Fernando earthquake, north-south component recorded at the Holiday Inn, Orion Avenue, Van Nuys (Building Report 29).

3. 1971 San Fernando earthquake, transverse (N.37°E.) component recorded at Bunker Hill Tower ground level.

Acceleration response spectra for these three ground motions, 3-percent damped, are shown in figure 19a.

DISCUSSION AND INTERPRETATION OF RESULTS

Comparison of Calculated Versus Code Forces

In the previous paragraphs, results of the dynamic analysis and subsequent comparisons of those results with code values were presented. The results of the dynamic analysis, in general, showed that the level of code seismic forces was less than that the structure was required to resist. If a comparison of base story shears and base overturning moments is made, computed dynamic story shears are 2.8 to 3.0 times larger and computed dynamic overturning moments are 2.5 to 2.8 times larger than corresponding minimum 1970 UBC values (figs. 17 and 18), even with $J = 1.0$ for code overturning moment computations. Examinations of computed member forces revealed that, except for a few members, member forces due to combined vertical and seismic forces were less than those expected to cause yielding and it was determined that members of the lateral force-resisting system did not suffer structural damage.

Correlation With Building Design Requirements

Because the analysis confirmed the linear-elastic response of the structure and because there is no significant yielding of columns or girders, it would appear that code seismic design criteria are adequate. However, on the basis of dynamic analysis results, code design seismic forces are seen to be much less than those experienced. Figures 17c and 18c show that the dynamic story shears are approximately 2.8 to 3.0 times minimum code values and that the dynamic overturning moments are similarly higher than the code values.

Theoretically, if the various structural elements had been designed to meet just minimum code seismic requirements, the dynamic loads in these members would have been more than twice the design values. For example, the ratios of M/M_p in table 7 and M/M_{pc} in tables 8 and 9 would then have been greater than 1, indicating yielding.

Examination of the design calculations revealed the reason for the discrepancy between anticipated values of M/M_p and calculated dynamic values. As a first step, the structure was designed in accordance with minimum code requirements, and all members were proportioned to meet the allowable working stresses specified in the code. Then a dynamic analysis of the structure was made using the ground motion of a known earthquake (El Centro, 1940,

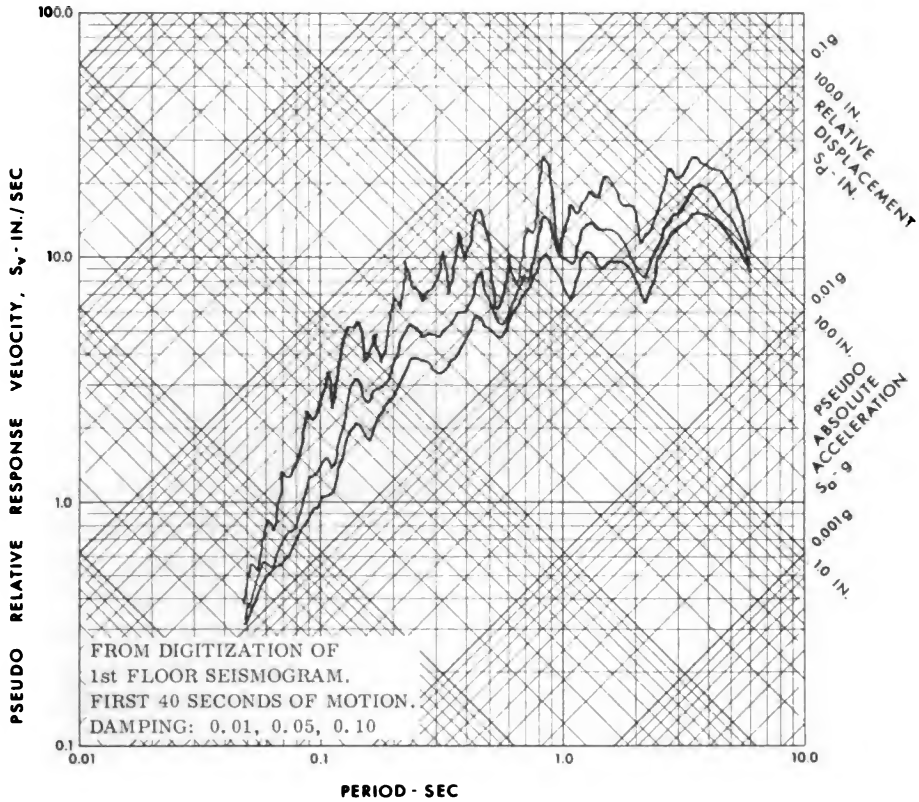

Figure 12.—Bunker Hill Tower. Transverse response spectra.

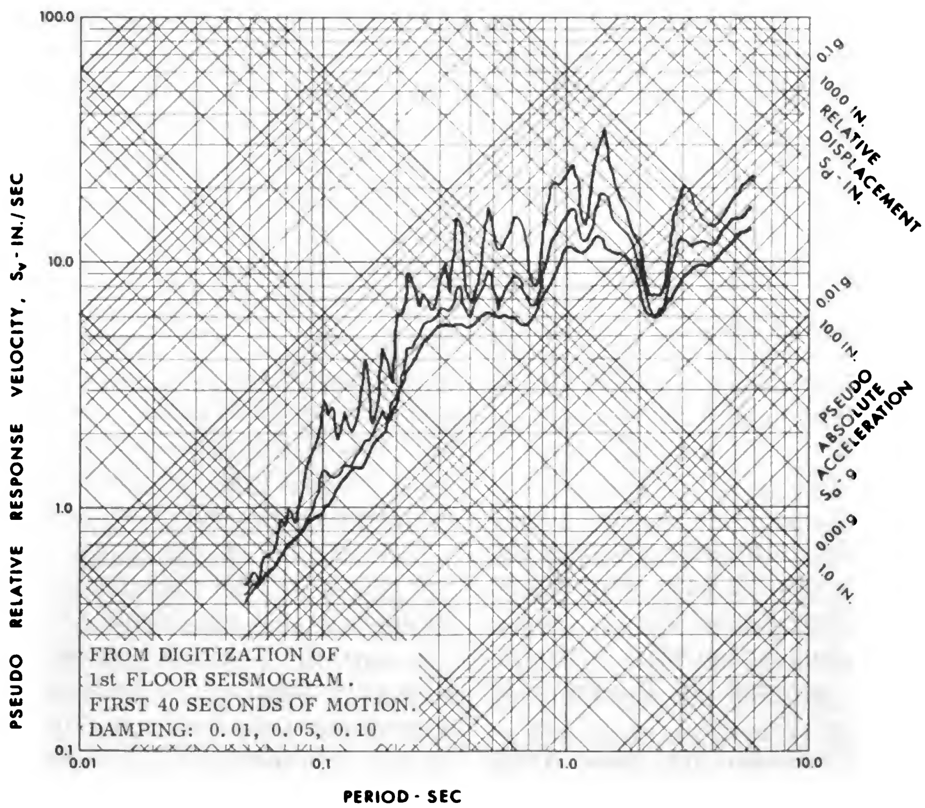

Figure 13.—Bunker Hill Tower. Longitudinal response spectra.

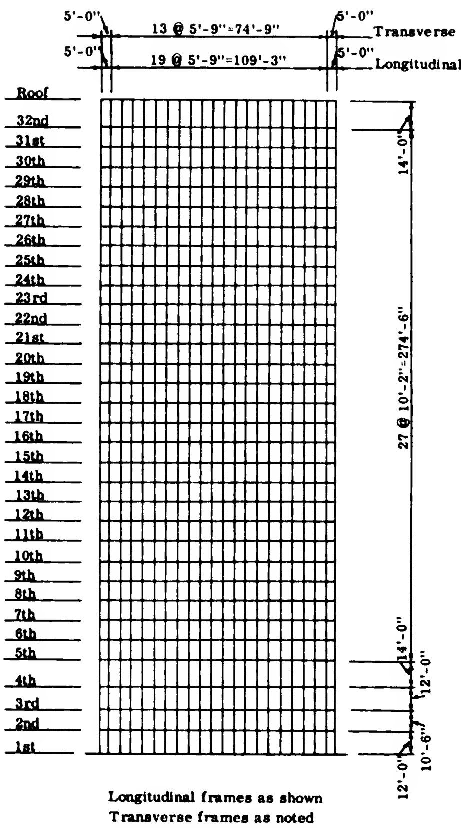

Figure 14.—Bunker Hill Tower. Elevations of exterior frames.

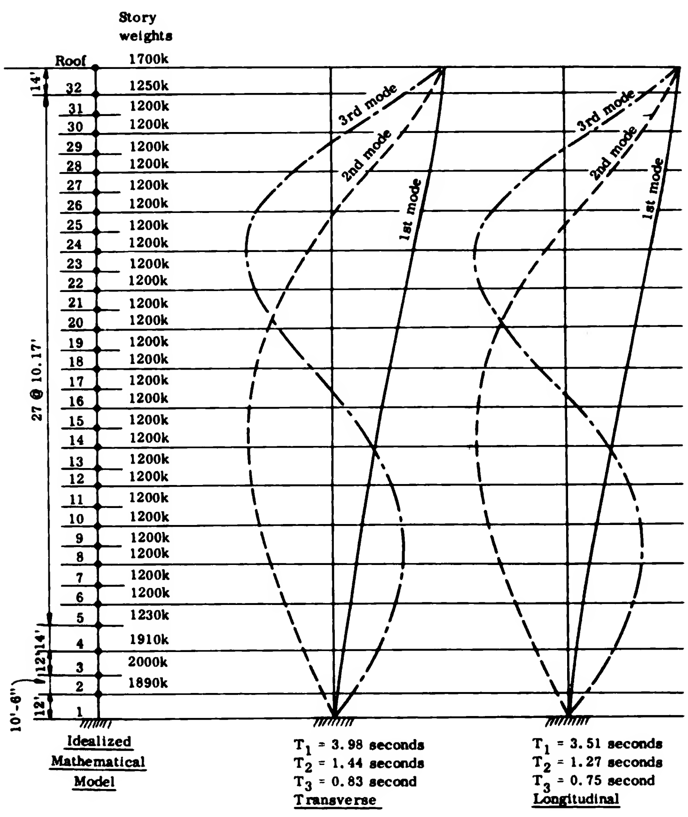

Figure 15.—Bunker Hill Tower. Calculated periods and mode shapes.

north-south component), and the resulting member forces were calculated and compared to the ultimate capacities of the members. Some member sizes were changed where dynamic stresses exceeded yield levels. In addition, the structural frame was designed to meet a prescribed drift index of 0.2 percent of the story height under design wind forces. Subsequently, most of the girder sizes and some of the column sizes had to be increased to bring the drift down to the prescribed limit.

As a result of all the requirements considered in the final design, the dynamic stresses in the structural steel frame for the San Fernando earthquake are largely within the yield strengths of the members, even though actual earthquake forces were much greater than the minimum code seismic forces.

Modal Analysis Procedures

To test the accuracy of the mathematical models an attempt was made to verify both mode shapes and periods. Mode shapes could not be confirmed by the recorded data because of the lack of a sufficient number of data points for this 32-story structure. Records of motion were obtained for only the roof and 16th-floor levels, and these would not provide a sufficient number of data points to verify calculated mode shapes, especially inflection points of the higher modes. Better results were obtained, however, when comparing the calculated and recorded periods.

The calculated fundamental periods for the bare structural steel frame are approximately 4.0 seconds in the transverse direction and 3.5 seconds in the longitudinal direction (fig. 15). Fundamental periods implied in the records of motion, which become apparent after 20 seconds of motion (fig. 16), are approximately 4.0 seconds in the transverse direction and 3.4 seconds in the longitudinal direction. This comparison provided some help in determining the accuracy of the mathematical models, but verification was complicated by other factors.

Because any one of several parameters could be varied to alter the correlation between calculated and measured periods, achieving approximate correlation of fundamental periods did not assure that the calculated mode shapes were in fact a true representation of the actual structure. In addition, the possible errors in digitization, as discussed earlier, were sufficient to prevent all but approximate determinations of recorded periods. Consequently, precise verification of computed periods and mode shapes by

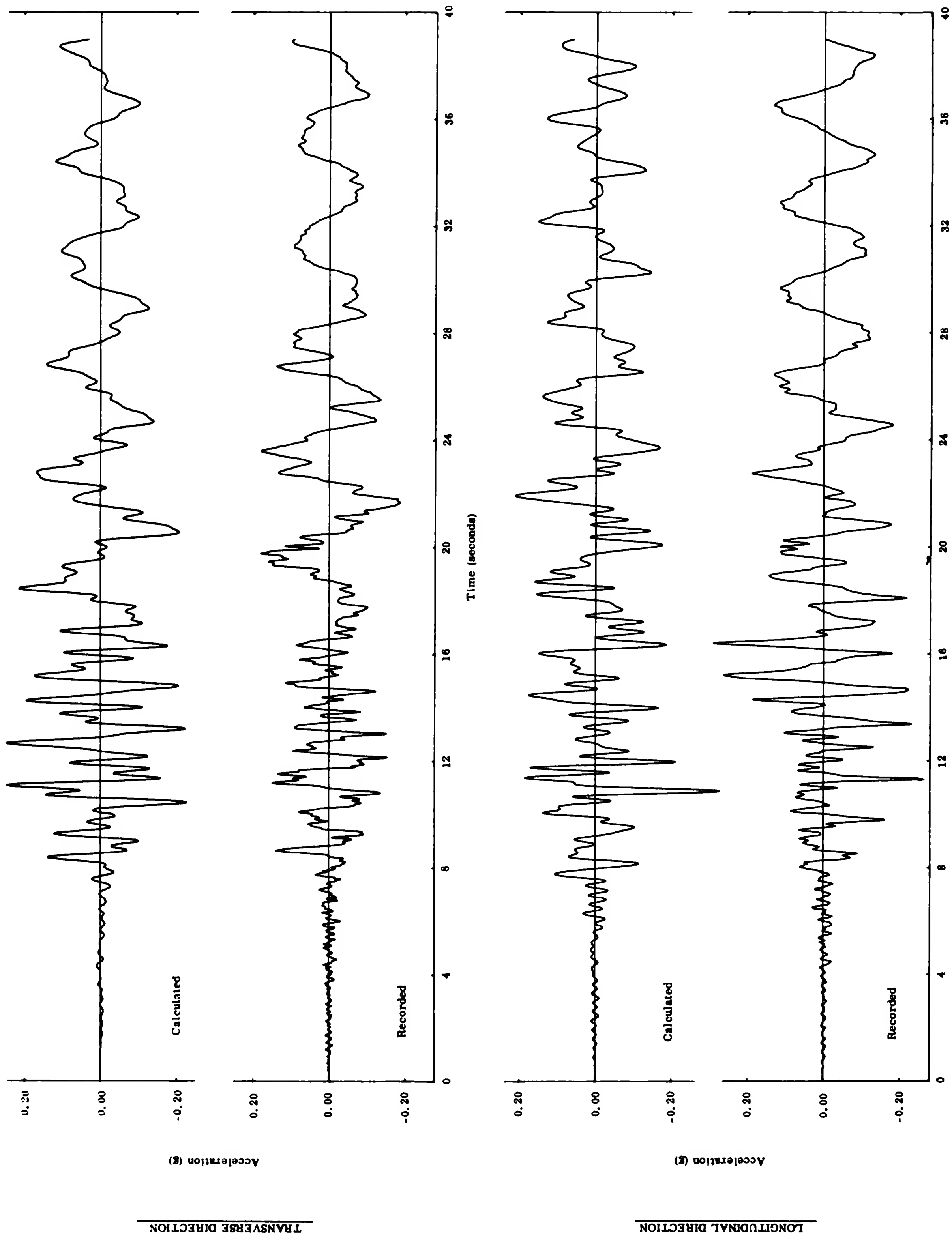

Figure 16.—Bunker Hill Tower. Calculated and recorded accelerations at roof level.

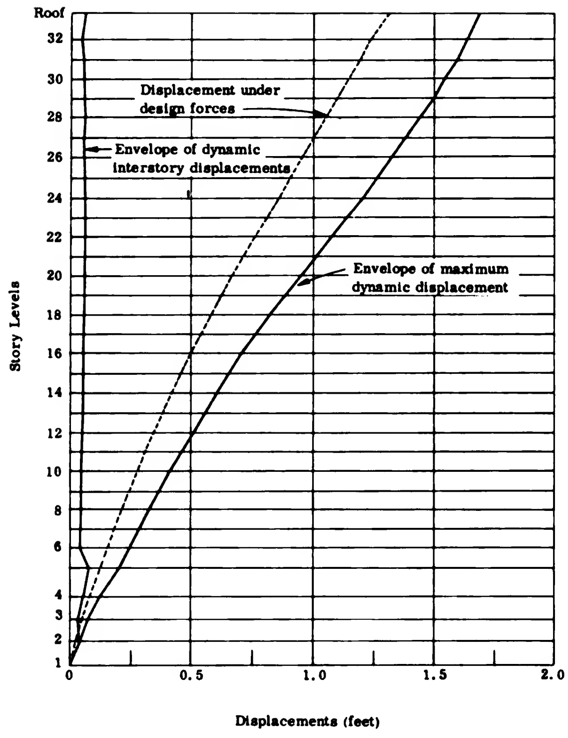

Figure 17a. TOTAL BUILDING DISPLACEMENTS AND INTERSTORY DISPLACEMENTS

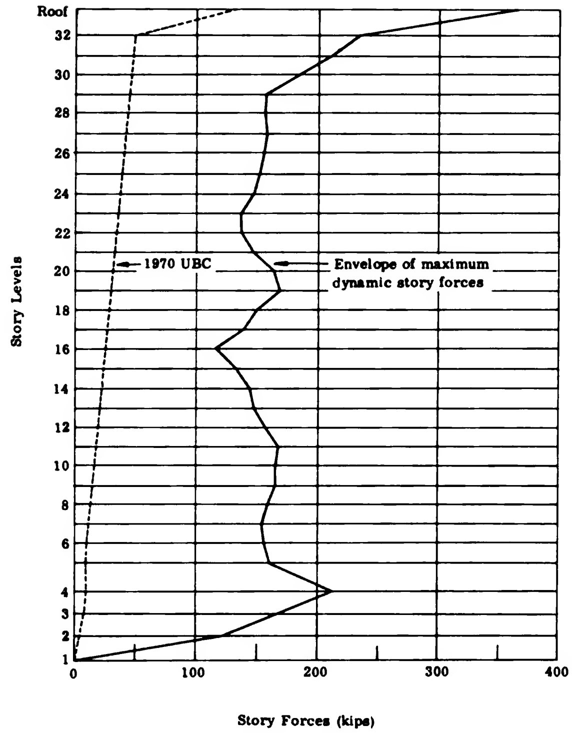

Figure 17b. MAXIMUM STORY FORCES

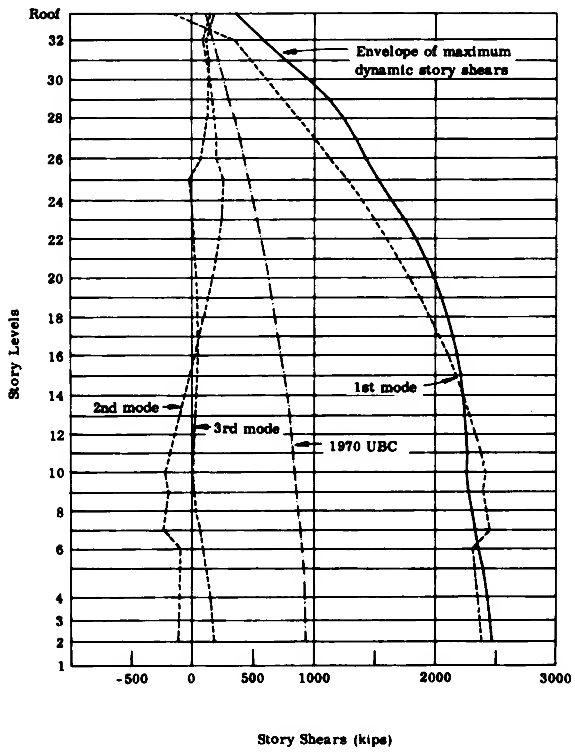

Figure 17c. MAXIMUM STORY SHEARS

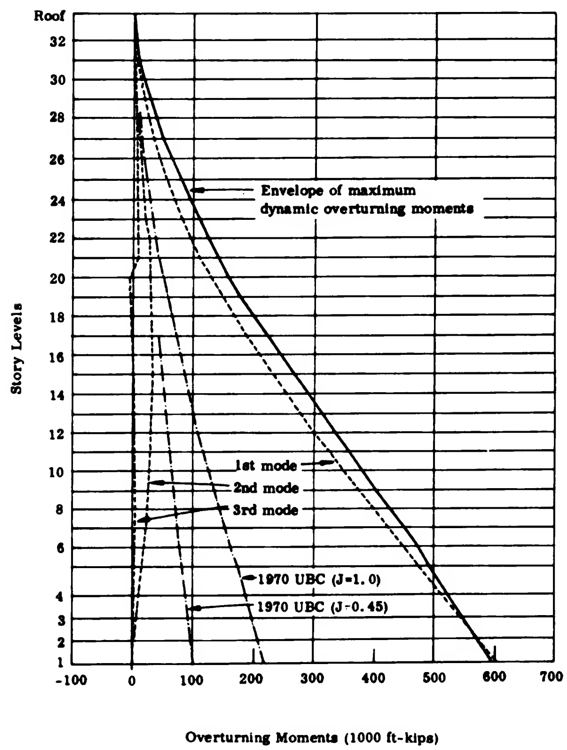

Figure 17d. MAXIMUM OVERTURNING MOMENTS

Figure 17.—Bunker Hill Tower. Dynamic response and design code values for transverse (north-south) direction.

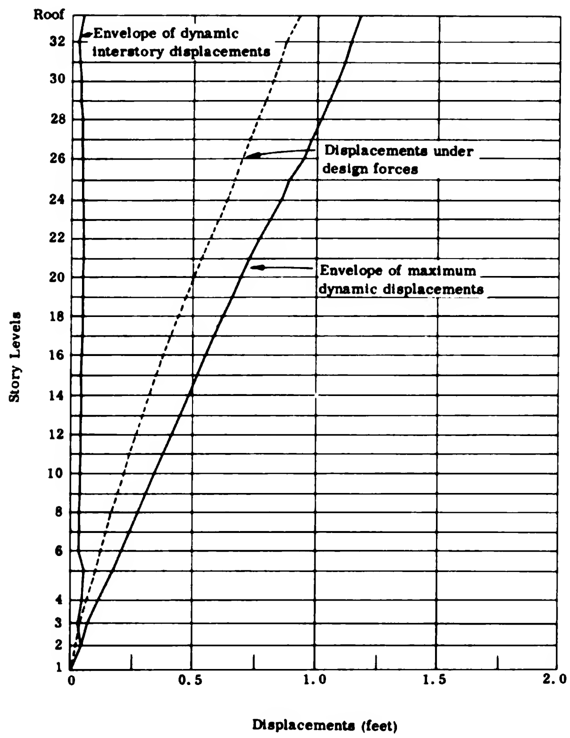

Figure 18a. TOTAL BUILDING DISPLACEMENTS AND INTERSTORY DISPLACEMENTS

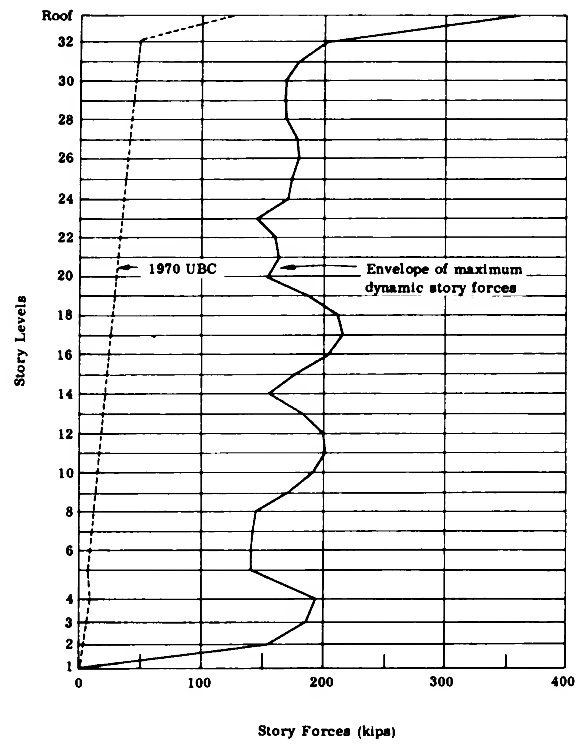

Figure 18b. MAXIMUM STORY FORCES

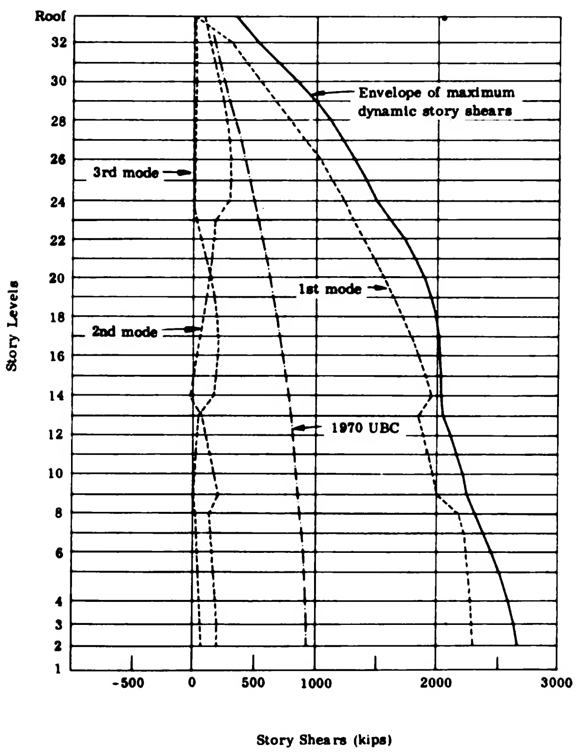

Figure 18c. MAXIMUM STORY SHEARS

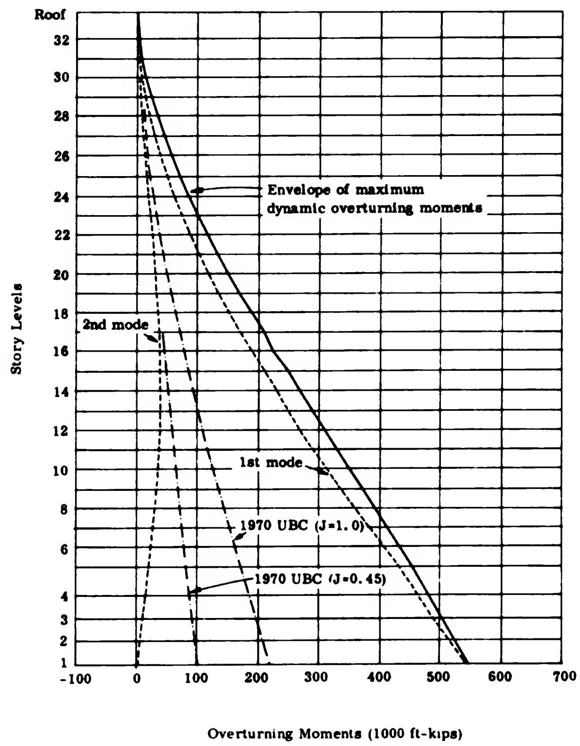

Figure 18d. MAXIMUM OVERTURNING MOMENTS

Figure 18.—Bunker Hill Tower. Dynamic response and design code values for longitudinal (east-west) direction.

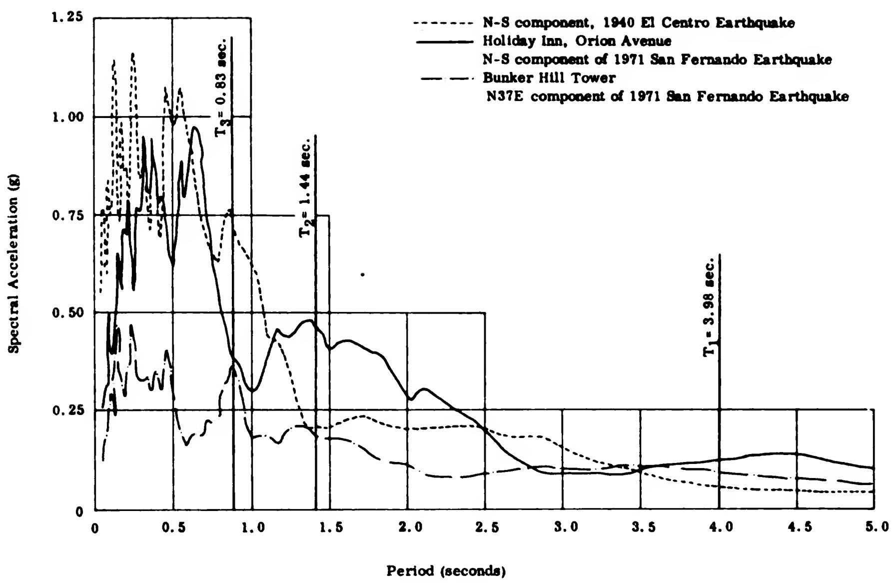

Figure 19a. RESPONSE SPECTRA (3% DAMPED) AND PERIODS OF VIBRATION FOR FIRST THREE TRANSVERSE DIRECTION MODES

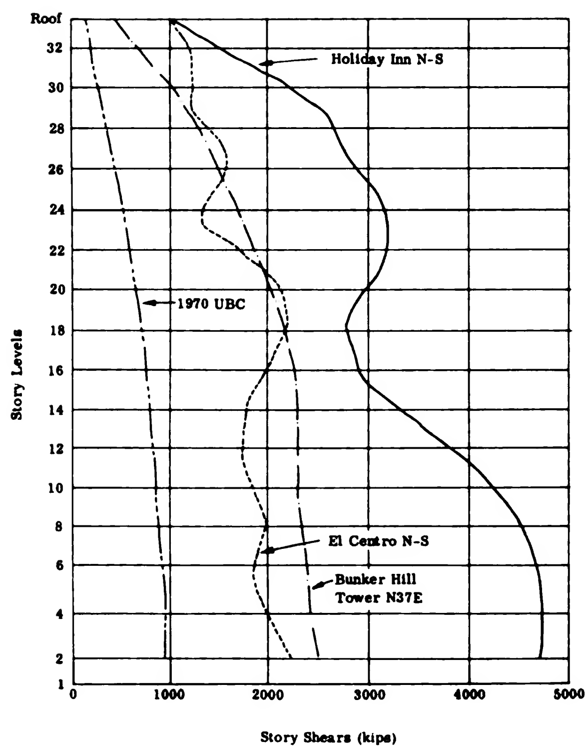

Figure 19b. MAXIMUM STORY SHEARS

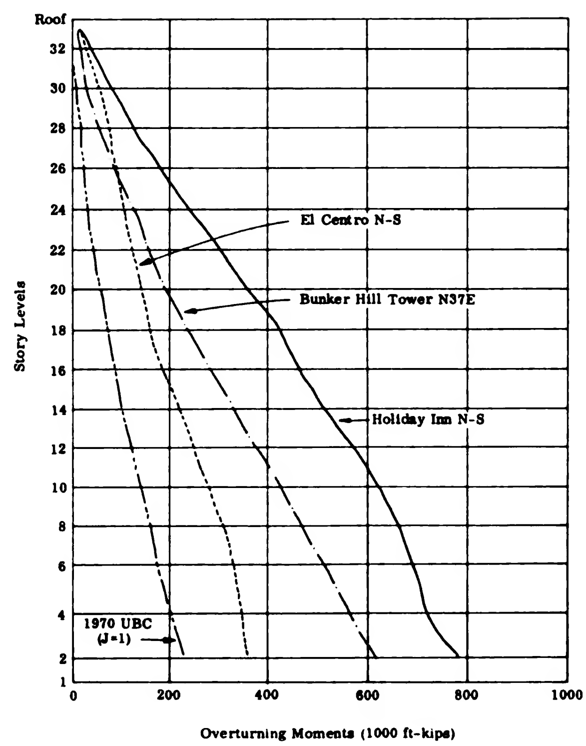

Figure 19c. MAXIMUM OVERTURNING MOMENTS

Figure 19.—Bunker Hill Tower. Comparison of effects of three different ground motions. Transverse direction.

data obtained from building-motion records was not possible.

Comparisons of Recorded and Computed Responses

Comparisons of recorded and computed responses were made by comparing acceleration time histories for the roof level. Both the general shape of the computed time history and the maximum values of computed acceleration response provided fair correlation with the recorded values as shown in figure 16 and in table 5. Better correlation of the amplitudes possibly could be obtained by adjusting applied damping values for each mode, rather than applying the same damping value to all modes.

Maximum computed accelerations do not occur at the same time intervals as the maximum recorded accelerations. Differences vary from 1 to 7 seconds. Most of the recorded peak accelerations occur between 10 and 15 seconds after the start of motion, but maximum calculated story displacements, interstory drifts, story shears, and overturning moments occur between 20 and 25 seconds after the start of motion. Inspection of the records of motion, however, offers an explanation for these apparent differences.

The response of the structure begins to build up after about 8 seconds of motion with the acceleration response consisting of a combination of modes. From 8 to 20 seconds after the start of motion, modes other than the fundamental mode predominate the response. After 20 seconds, the fundamental mode becomes the predominant mode. In this interval, the peak fundamental mode acceleration is not as large as peak accelerations associated with higher modes, but displacements associated with the fundamental mode (which are proportional to acceleration times fundamental period squared) are greater. Consequently, response to the earthquake resulted in the maximum forces occurring during the interval of response between 20 to 25 seconds after the start of motion.

Correlation With Damage Observations

Results of the dynamic analysis indicate that the occurrence of structural damage from this earthquake is unlikely. Member forces due to combined vertical and seismic loads are, with minor exceptions, less than yield capacities, but because most of the structural steel members and connections are covered with other building materials, this conclusion could not be verified by field observations. Reported damage was limited to architectural elements, principally walls and ceilings, and was considered minor. These observations tend to substantiate the assumption that no structural elements were damaged.

Vertical Accelerations

Vertical motion recorded at the roof and 16th-floor levels shows a considerable amplification over that recorded at ground level. As can be seen from table 2, an amplification of vertical ground accelerations of approximately 4 to 1 occurred at the roof as compared to the approximately 2 to 1 amplification found in both horizontal components. The increased vertical response was associated with a motion having a predominantly larger high-frequency content than that found in either horizontal component.

The effects of vertical ground accelerations were not considered in determining member forces. Although probably not in phase with maximum horizontal forces, vertical forces undoubtedly added to critical stresses in some members.

Effects of Different Ground Motions

Results of the study of the effects of three different earthquake ground motions indicated the importance of seismic exposure and site conditions.

Differences in seismic forces from different ground motions were large (figs. 19b and 19c). Base story shears were from 2.9 to 4.9 times, and base overturning moments were from 1.6 to 3.5 times, minimum 1970 UBC values with $J = 1.0$.

While epicentral distances and magnitudes varied considerably, the three ground motions are highly dependent on local soil conditions. The El Centro record was recorded on dense alluvium for a magnitude 7.1 earthquake at an epicentral distance of 7 miles. Motion at the Holiday Inn was recorded on moderately dense alluvium for a magnitude 6.4 earthquake at a 13-mile epicentral distance. The Bunker Hill Tower record was recorded on soft rock at an epicentral distance of 26 miles.

At the present time, the UBC does not have requirements to include local soil conditions or seismic exposure risks (except the very general seismic risk coefficient Z).

SUMMARY OF FINDINGS

Results of this seismic study, despite any possible errors in the digitization of recorded ground motion, yield the following findings:

1 The structure resisted the earthquake elastically, except for possible minor local yielding in a few girders and at the corner columns near the ground floor.

2 Actual earthquake forces were greater than prescribed code minimums.

3 Damage was slight and consisted of minor and localized cracking to walls and ceilings, three windows broken by falling objects, and temporary disruption of elevator service.

4 Results of the dynamic analysis indicate that no structural damage was to be expected, and this assumption was verified by limited field observations.

5 Story shears and overturning moments were determined to be highly dependent on the characteristics of the ground motion.

RECOMMENDATIONS

As a result of this study, several areas of possible change in seismic design practice and building instrumentation have been recommended. These are presented in the following paragraphs.

Minimum Seismic Code Requirements

One possible means of providing structures with greater earthquake resistance would be to increase the minimum code seismic requirements. If the present equivalent static force method of determining design seismic forces is retained, then an increased base shear and overturning moment requirement may reflect more closely the loading conditions that may be imposed on a structure.

Site-Specific Seismic Criteria

For some structures, especially high-rise buildings, a realistic approach to seismic design might require the use of dynamic analysis procedures utilizing one or more hypothetical design earthquakes based on the seismic exposure and soil conditions of the site.

Inelastic Frame Behavior

In order to insure the vertical load-supporting integrity of columns, lateral force-resisting frames should be designed so that any inelastic behavior initiates in and is confined to girders. Columns should remain essentially elastic at all times, and girder flange and shear connections should be capable of developing the full plastic moment capacity.

Architectural Elements

Walls, ceilings, and partitions should be designed for seismic movements. The amount of movement to be designed into the element should be based on maximum possible interstory drifts, rather than on deflections computed for the present level of code seismic forces.

Equipment and Other Building Contents

Elevator machinery, equipment, and other building contents should be secured against seismic forces and damage from seismic movements. Present seismic design practice often neglects equipment and building contents.

Strong-Motion Instrumentation

Future analytical studies of high-rise building earthquake behavior, especially verification of mode shapes, would be aided if a sufficient number of recording instruments were placed between the top floor and ground levels of high-rise buildings.